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ORDINARY MEETING.

15 November, 1938.

WILLIAM JAMES EAMES BINNIE, M.A., President, in the Chair.

The Council reported that they had recently transferred to the class of

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Harry Edward Whitehouse, M.C., M.A. (*Cantab.*).

And had admitted as

Students.

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David Arthur Gordon Allan.
George Kent Armstrong, B.E. (*New Zealand*).
Frank Douglas Ashton.
Denis Ayres.
John Eustace Bailey.
George Ball.
Jahar Lal Banerjee, B.Sc. (*Delhi*).
Norman Robert Banner.
John Barr.
Robert Alfred Barr.
Ronald Ernest Bartlett.
Ronald Batcock.
William Geoffrey James Bellis.
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John Cameron.
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Peter Chalmers.
John Rupert Charnley.
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Bernard Roland Cowles.
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Murdo Macleod.
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Walter Neville Watterson.
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Frederick Wright, B.Eng. (*Liverpool*).
George Stanley Young.

The following Paper was submitted for discussion, and, on the motion of the President, the thanks of The Institution were accorded to the Author.

Paper No. 5167.

"The Principles of River-Training for Railway Bridges, and their Application to the Case of the Hardinge Bridge over the Lower Ganges at Sara." †

By Sir Robert Richard Gales, F.C.H., M. Inst. C.E.

TABLE OF CONTENTS.

Part I.—Introduction and Examples.

Part II.—General Principles.

† Correspondence on this Paper can be accepted until the 15th March, 1939.—Sec. Inst. C.E.

Part III.—The Training Works of the Hardinge Bridge.

Part I.—INTRODUCTION AND EXAMPLES.

Historical.

The bund-and-apron method of control of rivers in the vicinity of railway bridges was introduced in 1888 by Mr. James Richard Bell at the construction of the bridge over the Chenab at Sher Shah, of which he was the engineer-in-chief. Of this work, Mr. F. J. E. (later Sir Francis) Spring was executive engineer, and the Author one of the assistant engineers. The Author's principal contribution to this system of river-training was the introduction of the curving back of the head of the Bell bund to such a degree as to ensure that any bend of the river, at its deepest embayment towards the railway approach-bank, would encounter a continuous stone-pitched bank offering no interference with smooth flow, instead of meeting an obstruction in the form of a spur as was the case with the original slightly-curved head. This idea was carried into effect in the design and construction of the bridge over the Ganges at Allahabad, as described in the Author's Paper on the Curzon bridge,[1] the name by which the bridge was subsequently known. The bridge was under construction and the earthwork of the Bell bund was nearing completion early in 1903, when Mr. Bell, then a cold-weather visitor, and Mr. Spring, at that time writing his Technical Paper,[2] together with the Author, made a joint inspection of the work, which was not without effect in determining some of the principles of river-training on the Bell bund system. Following, as he believed, Bell's practice, the Author had made the face-slope of the bund 1 in $1\frac{1}{2}$, and had arranged to put the reserve stone on the upper part of the slope with a face-slope of 1 in 1, with the idea that the reserve stone would be brought into action earlier if the apron stone proved to be insufficient, and slips occurred in the bank. Mr. Spring maintained that the stone pitching on the face of the bank should never be disturbed and that the slope should be 1 in 2, and it was thereupon agreed that aprons should be designed, as far as possible, so that slips should not extend into the slope of the bank, and that the slope covering should be designed in the expectation that it would remain as the permanent protective covering of the slope.

[1] Minutes of Proceedings Inst. C.E., vol. clxxiv (1907–8, Part IV), p. 1.
[2] "River Training and Control." Technical Paper No. 153, Simla, 1903.

The curved-back head was then considered and approved, and drawings were supplied which were reproduced in Spring's work on river-training.[1] The adoption of the principle necessitated a good deal of revision of parts of Spring's work, for which proofs were subsequently supplied to the present Author for verification. This form of head has lately been referred to as the Curzon-Bridge type head, or Curzon head, on account of the confusion arising from its being spoken of as a curved-back head which does not sufficiently distinguish it from a slightly-curved head.

The Bund-and-Apron or Bell Bund System.

The first exposition of the Bell bund system is contained in one of the early Technical Papers [2] published by the Government of India, and as it is of general interest, it has been reproduced as an Appendix to the present Paper. The Bell bund method of dealing with alluvial rivers made permissible a considerable reduction in the length of the bridge, and, by providing permanent banks where there were none before, ensured the stabilization of the course of the river through the bridge. As soon as the success of this method at the Chenab bridge at Sher Shah became known, it was realized that this was an epoch-making conception, and the construction was put in hand of a number of large bridges which had previously been in abeyance owing to the difficulty in coming to any logical conclusion as to the length of bridging and depth of foundations required, and to the uncertainty of being able to guide the river through the bridge by means of the spur system of training previously in vogue. The Bell bund method not only reduced the first cost of the bridge proper, but, what was of more importance, it reduced the cost of maintenance of the training works, which was such a formidable item with the spur system. In short, it made possible the construction of permanent and workmanlike bridges in situations where this had not previously been found possible.

All honour to James Richard Bell!

The Guide-Bank System.

Bell's Technical Paper [2] was followed towards the end of 1903, thirteen years later, by Spring's Technical Paper.[3] In this the nomenclature was revised, "bund-and-apron" or "Bell bund" becoming "guide-bank," and the experience which had been gained on the Bell bund bridges already built or building was collated and recorded. This publication contains a mine of information and is the standard text-book on the subject. The Author of the present Paper, having lately read it through, was interested

[1] *Loc. cit.*

[2] J. R. Bell, "The Continuous Bund and Apron Method of Protecting the Flanks of Bridges for Rivers in the Punjab (India)." Technical Paper No. 2B, Simla, 1890.

[3] Footnote (2), p. 137.

to find, on a point of nomenclature, that the word "eddy" is not used by Spring, its place being taken by "swirl." This has been the cause of a great deal of misunderstanding, and it is suggested that where used in connexion with river-training the word "swirl" should be taken to mean a moving eddy.

The cost of continuous stone-pitched guide-banks is too great for the system to have much future in training long stretches of open river. It has, however, been used by Bell with complete success in training the river entrances of seaports, as on the Karnafuli at Chittagong and to a limited extent at fixed points, such as at weirs for irrigation purposes. In country with rivers which can only be bridged with the aid of the guide-bank system, there are either no roads, or roads with boat bridges or ferries, in which case combined road and railway bridges have been built. It thus comes about that it is in connexion with railway bridges that the advantages of the guide-bank system have been demonstrated. The advantages are greatest in the case of a railway bridge because the flanking guide-banks properly employed not only safeguard the bridge, but, within limits, ensure the safety of the approaches, which is the vital consideration for a transport undertaking.

Description of Country in which the Guide-Bank System has been Used.

India is divided into two parts. The southern part, or Deccan, formed of the older plutonic rocks, is separated from the more recent Himalayas by a depression filled in earlier times by an arm of the sea connecting the Arabian Sea with the Bay of Bengal. The other part, or northern India, comprises the great alluvial plains, formed by the filling of this depression by alluvial deposits brought down mainly by the rivers issuing from the Himalayas, which extend from west to east across the north of India.

It is in the alluvial plains of northern India that the guide-bank system has been used with varying degrees of success. The bridges have been, for the most part, over affluents of the Indus, the Ganges and the Brahmaputra, but the Ganges itself has been bridged. The conditions are usually such that, owing to the depth of the alluvium, the piers are carried on wells sunk to their full depth and founded in sand. The only stabilizing influences are the thin beds of *kunkur* which occur sporadically throughout these great alluvial plains. Before the general adoption of cement *kunkur* formed the base of the lime mortars used in northern India. It is produced by the percolation of lime-bearing water, and the formation varies from small pebbles to thin beds of rough limestone. Thin beds of *kunkur* are often met with in sinking well-foundations, and such *kunkur* is often found to be the underlying cause of unexpected stability in some reach of a river. The Punjab rivers in their middle courses may be found to run at a lower level than the surrounding plain, and to confine their oscillations to

a limited width of country, but in the more recent parts of the hydraulic fill, such as the lower part of the Gangetic plain and the Brahmaputra valley, the rivers may be found to be running in self-raised elevated channels, out of which they break to begin building up a fresh part of the country on an entirely new course.

Classification of Rivers to which the Guide-Bank System is Applicable.

Rivers to which the guide-bank system of training is applicable may be classified as follows :—

Class (*1*). This consists of rivers running in an alluvial plain or *khadir*, between what may be described as permanent banks. The *khadir*, in which the river is free to wander, may be much wider than is necessary to take the flood-discharge, and considerable saving in cost is obtained by bridging only a fraction of the width of the *khadir* and providing a guide-bank or guide-banks flanking the bridge. It was held by Bell, from experience chiefly of the Punjab rivers, that his smooth-flow flanking bunds or guide-banks required no further works upstream, and economy was claimed for this in comparison with the spur system, which might include isolated works at great distances upstream where proper supervision was difficult or impracticable. In rivers in class (1) this contention has been completely established. It is in this class of river, moreover, that the principle of proportioning the upstream length of the guide-bank to the length of the approach-bank to be protected can be employed to the best advantage.

Class (*2*). This class also consists of rivers which run in a *khadir* that is much too wide for them. Although the banks can by no means be described as permanent these rivers do not encroach upon them to a great extent, and it may be that occasional hidden beds of *kunkur* have some influence in controlling the width of the *khadir*. In this class are the Punjab rivers, particularly in their lower courses before joining the Indus, of which the Chenab at Sher Shah and the Sutlej at Adamwahan may be taken as examples. In a recent Paper by Colonel William Macrae [1] is described the application of the guide-bank system to the shortening of the Empress bridge over the latter river at Adamwahan, which was built before the guide-bank system was introduced; many bridges in the Punjab have been similarly treated. In a number of these the success of the shortening has been much assisted by partial stabilization of the river, brought about by detached works previously carried out upstream. This is notably the case at the Alexandra bridge over the Chenab at Wazirabad, where such detached works are to be found to a distance upstream of $3\frac{1}{2}$ miles.

[1] "Training-Works in Connection with the Shortening of the Empress Bridge over the River Sutlej." Minutes of Proceedings Inst. C.E., vol. 237 (1933–34, Part 1), p. 119.

Class (*3*). The third class of river to be bridged is that in which there are no banks at all and the river is free to wander to an unlimited distance in either direction. To this class belong the cases where there is one permanent bank and the river is free to wander to an unlimited distance in one direction only. The problem in this class is one for which no general solution has yet been found. It is, indeed, only by building the bridges first and finding by experience the difficulty there is in maintaining them that it has been realized that this class of river provided a special problem.

It has been observed that rivers have a tendency to keep to a deep channel such as should be found between the flanking guide-banks of a bridge, and the attraction of a deep channel is a material factor in the success of the guide-bank system. In addition, the life of a bridge over a river of this class can be prolonged by the use of longer guide-banks and by taking advantage of any natural features, such as beds of *kunkur* or indurated clay, which investigation may bring to light.

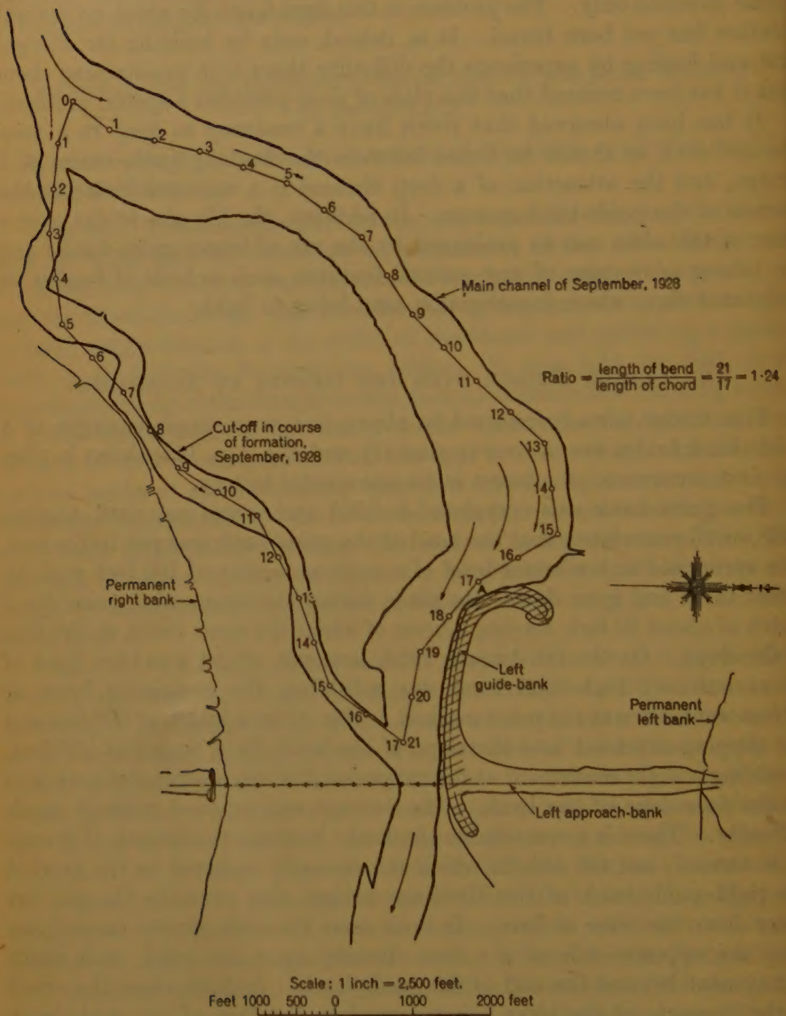
The Curzon Bridge over the Ganges at Allahabad.

This bridge, already referred to above, is a very simple example of a guide-bank bridge over a river in class (1), and *Fig. 1* (p. 142) shows in plan the first occurrence of interest in its uneventful history.

The guide-bank was completed in 1903 and it was not until August, 1928, or 25 years later, that the head of the guide-bank was put to the test. The apron laid at low-water level of a uniform section of 100 feet wide by 4 feet thick had gone down into place during the intervening years for a width of about 50 feet, leaving a berm of about the same width at the foot of the slope. On the 1st August, 1928, however, whilst a sudden flood of about ordinary high-flood level was subsiding, the remaining berm of 50 feet went down at the point marked A (*Fig. 1*) for a length of 500 feet and the slipping extended into the slope of the bank for a length of 150 feet, involving a slight movement at formation-level of the reserve stone stacked on the face-slope of the bank. The damage was repaired without much difficulty. There is a remarkable similarity between this attack, if it may be so termed, and the attacks which subsequently occurred on the head of the right guide-bank of the Hardinge bridge, also over the Ganges but lower down the river at Sara. In both cases the main stream came down from the opposite side of the river directly upon the head, with slight embayment beyond the end of the guide-bank. In both cases the effect of the pressure of the river being normal to the face of the guide-bank head appeared to bring about more damage than would ordinarily be caused by the river running tangentially to the curve. An examination with regard to the sufficiency of the apron-stone shows that the deepest bend-scour sounded before and during the construction of the Curzon bridge was 30 feet below low-water level, and to this 20 feet were added for contingencies, making 50 feet, and the apron of 100 feet by 4 feet gave 400

cubic feet of stone per foot run. This is equivalent by Spring's diagram for the design of aprons to the quantity required for a scour of 40 feet, or

Fig. 1.



THE CURZON BRIDGE OVER THE GANGES AT ALLAHABAD.

an addition of 33 per cent. to the greatest depth of bend-scour previously ascertained. This addition would bring the bend-scour at a soft bank up to the bend-scour at a hard bank, and an allowance for contingencies at a guide-bank head of 50 per cent. would bring the depth of scour to be pro

vided for up to 60 feet. The depth of scour actually found at point A in January 1929, some months after the flood had subsided, was found to be 45 feet below low-water level, and the provision for 60 feet at the head and 50 feet for the rest of the guide-bank would therefore probably not be excessive.

The occurrence described above exemplifies and confirms that a normal attack on the head of a guide-bank has a more damaging effect than any smooth-flow oblique attack which can come upon the intermediate part of the guide-bank, and it emphasises the need of increased protection for the head in all cases.

From the example of the Curzon bridge, consisting of fifteen spans of 200 feet, may be deduced also the relation between the length of the guide-bank and the length of the approach-bank which can be protected by it. In Figs. 2 (*a*), Plate 1, the width of the waterway denoted by L is taken as the length of the bridge less half the two end spans, which are usually obstructed by the guide-banks. The distance to which the guide-bank extends upstream is made equal to the waterway L. The sharpest bend of the main river to be found in the vicinity has a radius of 1,500 feet, and it is evident that a bend of this radius, as shown at the back of the guide-bank, cannot threaten the approach-bank. In other words, the length of guide-bank which was sufficient to ensure no serious obliquity of current at the bridge-piers is obviously more than sufficient to ensure the safety of the approach-bank. In Figs. 2 (*b*), Plate 1, the development of a similar guide-bank bridge in the middle of a *khadir* between two permanent banks indicates the greatest length of approaches which can be protected by guide-banks of length L. The radius of the bend shown at the back of the guide-bank is 3,300 feet, which was the radius of the bend in the main river near the bridge-site in the year 1902. This bend is assumed to cut-off at a ratio of 1·75 as shown by the dotted line, with a margin of safety of $L/3$ between the bend and the approach bank. The ratio of 1·75 taken for the cut-off in this diagram is the ratio at which cut-offs are found to take place in the Lower Ganges, but in actual practice, in order to determine the appropriate length of the guide-bank, it would in all cases be necessary to ascertain the ratio of bend to chord at which cut-offs occur in the river it is proposed to bridge. It will be seen from the diagram that in the example taken, a river of width $L = 2{,}800$ feet running between permanent banks at a distance apart of $7\,L = 19{,}600$ feet (approximately 3·75 miles) can be bridged safely with guide-banks of length $L = 2{,}800$ feet. Similarly, in Figs. 2 (*c*), Plate 1, assuming a similar development for guide-banks of length $2\,L$, it would appear that the width between permanent banks for which the bridge-approaches would be protected would extend to $13\,L = 36{,}400$ feet (approximately 7 miles).

The general principle to be deduced from the above is that the length of the approach-bank protected by a guide-bank is proportional to the length of the guide-bank.

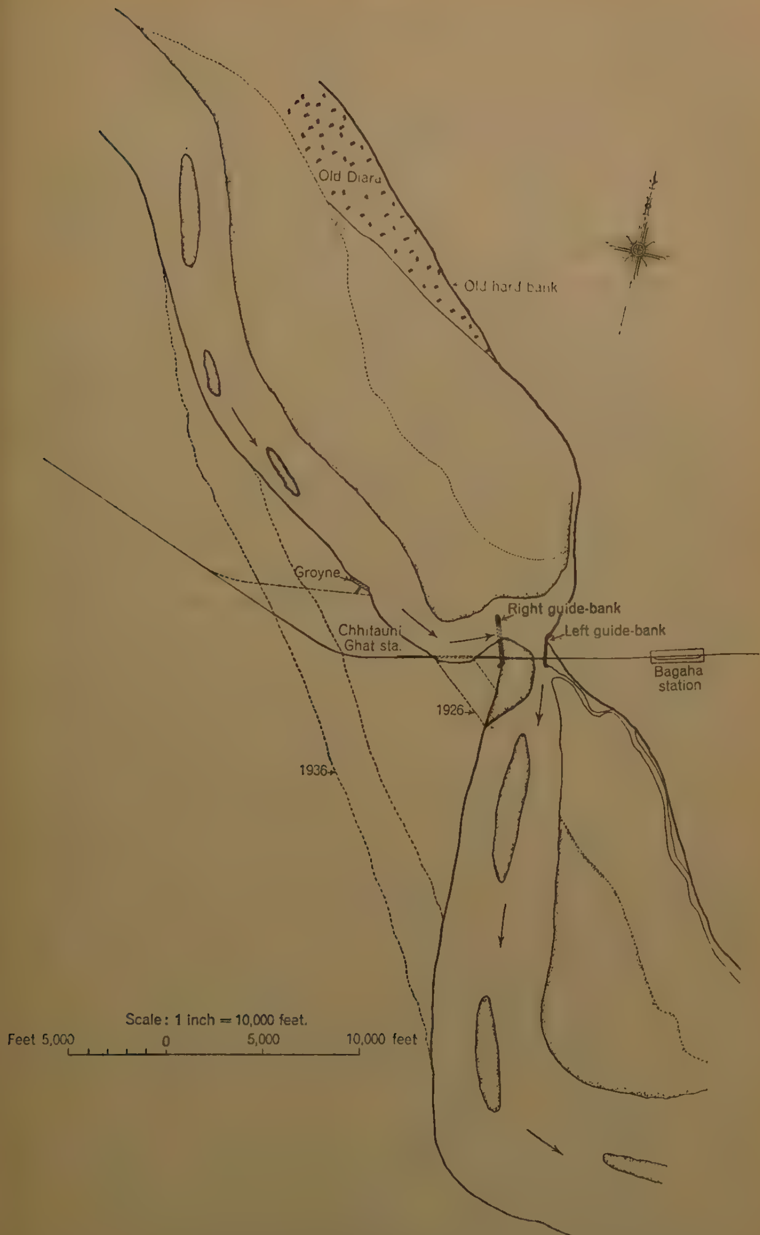
It should be mentioned here that, although the ratio of bend to chord at which cut-offs occur varies for different rivers and the ratio can usually be found from previous surveys for any particular river, the cut-off does not always take place at what seems to be the obvious place on the plan; by referring again to *Fig. 1*, it will be seen that before any further development of the embayment at the head of the guide-bank took place a cut-off from a point 9,000 feet (1·75 mile) upstream, with the low ratio of 1·24, became clearly indicated. The attack on the head of the guide-bank described above took place in 1928, and by 1933 the cut-off was completely established. A similar case occurred at the Sutlej bridge at Ferozpur where, between 1900 and 1901, and starting from a point 16,000 feet (3 miles) above the bridge, an extensive cut-off with a chord 13,000 feet (2·5 miles) in length and a ratio of 1·75 became fully established in a single year in preference to comparatively minor cut-offs at the guide-bank head with the tempting ratio of 2·25. Such extensive cut-offs are easily explained. A river in high flood will always take as straight a course as possible and if the low-water channel is fairly well centred at a distance of 2 or 3 miles above the bridge the flood-water in passing over sandbanks which are free from vegetation will usually leave a good low-water channel. If, however, the river above the bridge has moved away laterally from the axis of the bridge so as to advance towards the approach-bank at the back of the guide-bank, the cut-off will usually take place at the normal ratio for that river, although this conclusion may be modified if the chord has become impeded by coarse grass or scrub.

On referring again to Figs. 2 (*b*), Plate 1, it will be seen that three different courses marked B, C and D are shown to indicate the different courses which the main stream may be expected to take on leaving the head of the guide-bank in different stages of embayment towards the approach-bank. Course B is that taken in 1928 as appears in *Fig. 1*, when the pressure of the main river coming down upon the head from the opposite bank kept the main stream in contact with the face of the guide-bank down to the bridge. Courses C and D indicate, as found by experience at the Elgin bridge over the Gogra (an affluent of the Ganges), that the main stream will leave the curved head tangentially in a direction depending on the extent of the embayment, and course D should not be taken to indicate an extreme position. At this bridge, on a course corresponding to course C, no trouble appears to have been experienced from eddy action at the point marked C_1, where the main stream leaves the curved head tangentially; but an eddy set up at C_2 has been observed on a falling river to have the effect of deepening the channel along the guide-bank.

The Bagaha Bridge over the River Gandak.

This bridge (*Fig. 3*), is taken as an example of a guide-bank bridge over a river which falls into class (3), and its history as known to the Author

Fig. 3.



GANDAK RIVER AT BAGAHA.

has contributed to the evolution of the guide-bank system and to some appreciation of its limitations. It is the only case known to the Author of the river breaching the approach-bank and short-circuiting the bridge. The bridge consisted of fifteen spans of 150 feet on wells sunk to a depth of 80 feet below low-water level. The river Gandak is an affluent of the Ganges, with a maximum discharge of 300,000 cusecs. The bridge was sited at about 30 miles from the exit of the river from the foothills of the Himalayas. It will be seen by reference to *Fig. 3* that the left bank is described as "old hard bank," and when the river was running alongside this bank it passed axially through the bridge. The right guide-bank was of the straight-ended variety and slightly longer than the bridge. The river, however, did not continue to run alongside the old hard bank but began to make its way westward, forming an embayment behind the right guide-bank, round the nose of which the whole river passed across to flow through three spans at the left abutment, scouring out pier No. 14 and destroying the right guide-bank. This was followed by the breaching of the western approach, and a year later the river broke through on a line between the groyne and the tail of the guide-bank, making the short circuit of the bridge complete. As the river was still pressing in towards the western approach, flooding the country so as to produce an afflux of 6 feet at a point on the railway line 7 miles distant from the bridge, projected repairs to the bridge were abandoned and the break in communications was accepted.

During the flood of the 24th September, 1924, there was a surface-fall of 4 feet at the left guide-bank between points 400 feet above and 400 feet below the bridge and a surface-velocity of 17 feet per second. In August 1924, with water-level 3 feet below the maximum reached in September, the fall between the guide-banks was 3·3 feet per mile and the current-velocity was 12·5 feet per second. The fall per mile of the country is not known, but the fall of 1·35 foot per mile of the unobstructed river on which the project was based might have been arrived at by observations in a "pool" during the dry season, when the succession of pool and rapids is more defined. During any sustained high flood running an unobstructed course, without much curvature, in open country, it would seem that the surface-fall of the river would be the same as the fall of the country. It is important to establish the rate of high-flood fall in an unobstructed river beyond which it would be unwise to proceed with a scheme for a guide-bank bridge, but although no very precise information can be gained from this example it may be inferred that the figure is something well below 3·3 feet per mile.

The afflux of 6 feet mentioned above at a point on the railway-line 7 miles distant from the bridge shows that nothing would have been gained by putting the bridge on a tangent crossing the fall of the country at right angles, as the afflux would then have been much greater. This consideration appears to confirm the conclusion that, at least in the case of rivers which spill over their banks to flood the country, the unobstructed

flood-fall of the Gandak at Bagaha is too great to be dealt with by the guide-bank system.

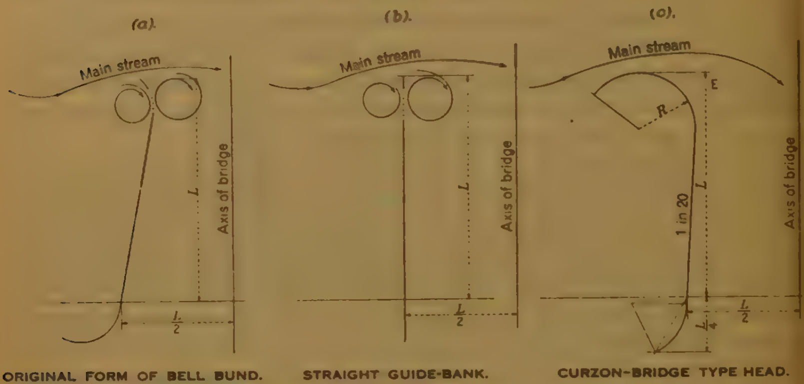
The straight-ended right guide-bank used at this bridge was a form devised to allow the extension of a guide-bank which could not be completed in one working season. This form originated at the Garhmuktesar bridge over the Ganges, where the exposed straight guide-bank, much shortened in length, appears to have reached stability after the pattern of a groyne with a long sloping nose of solid stone. The Bagaha right guide-bank is stated to have failed by percolation at the junction with the old temporary head, where a lot of *kunkur* and sausage protection had been thrown in, thus anticipating its failure by eddy-action.

Owing to the difficulty in many cases of carrying-out the construction of the whole of a guide-bank in one working season, the idea of making a part of it, fully protected, in one season and extending it the next has often been considered, and if the upstream end, temporarily protected by continuing the slope-stone and apron around it, had not been attacked during the intervening floods the temporary pitching stone could no doubt be removed and the guide-bank extended. If, however, the end had been attacked, either the extension would have had to be carried out probably across a new position of the main stream, which would be impracticable, or, if only local scour had occurred (during which the apron had dropped into the scour-hole), the extension would enclose, to the great danger of the guide-bank, an underwater slope covered with pitching stone through which water would have no difficulty in passing, either under the head of the afflux at the back of the guide-bank in times of flood or of impounded water during the subsidence of the floods. These considerations have hitherto deterred the Author from attempting the construction of a guide-bank in two parts, but there are occasions when this could safely be done with the aid of precautionary measures suitable to the circumstances of the case.

Figs. 4 (p. 148) illustrate the evolution of the form of guide-bank now generally adopted. *Figs. 4* (*a*) indicate the manner in which the original form of guide-bank failed at the Chenab bridge at Sher Shah. The main stream embayed at the back of the guide-bank and, crossing the head, set up eddies as depicted which breached the bank at the place shown by the dotted line, thus isolating the solid stone head which became submerged and disappeared. It is reasonable to believe that the straight guide-bank, *Figs. 4* (*b*), would fail in the same way. It may here be observed that when the main stream runs across the end of the guide-bank as in *Figs. 4* (*a*) and *4* (*b*) the guide-bank becomes a spur. A spur may be defined as any solid projection from the river bank into running water, which is the cause of stationary eddies. The spur form, intentional or unintentional, is the cause of most of the difficulties which afflict river-training. The Curzon-Bridge type head (*Figs. 4* (*c*)) represents an attempt to obviate the formation of dangerous eddies, and the eddy which may occur at point E with the main stream in the position shown has hitherto not proved dangerous.

From the foregoing examination of the conditions at the Bagaha bridge the conclusion has been reached that the site selected was unsuitable, and that, even if the bridge had had more waterway, deeper foundations, and a Curzon-Bridge type right guide-bank of greater length, and had been put

Figs. 4.



TYPES OF GUIDE-BANK.

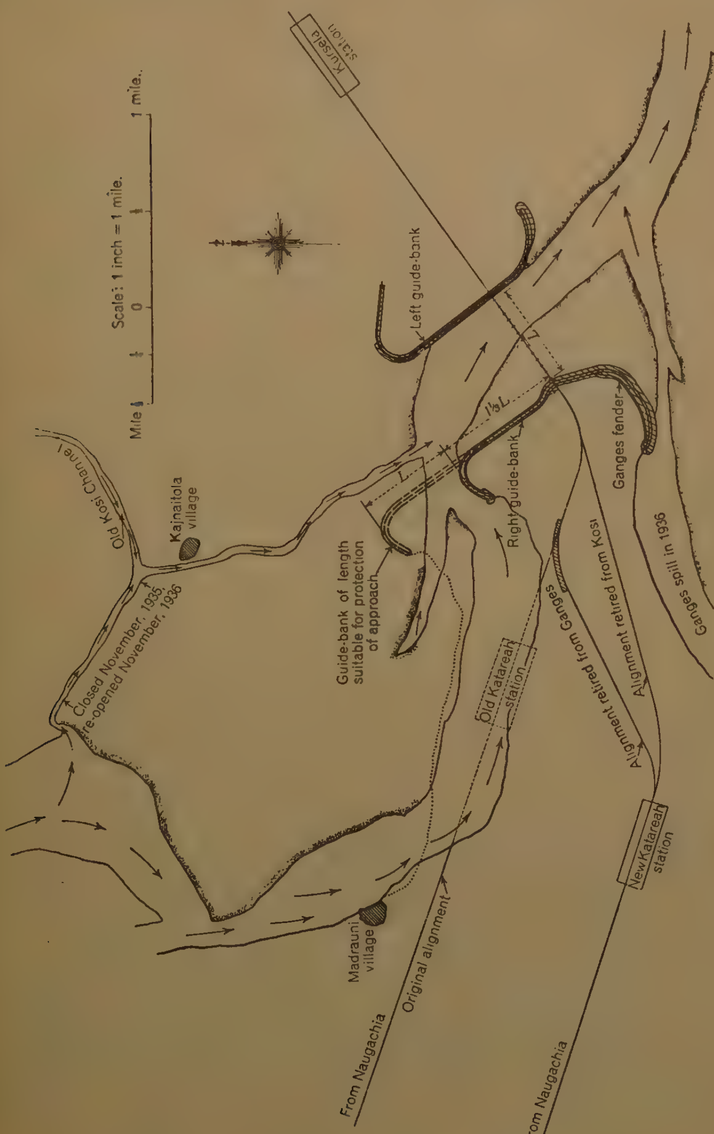
on a tangent crossing the natural fall of the country at right angles, it would not have been possible to maintain the western approach against an embayment at the back of the right guide-bank owing to the excessive flood-fall of the river, accompanied, as this must be, by afflux at the main-line bank and by turbulence of the river.

THE BRIDGE OVER THE RIVER KOSI.

As a second example of a class (3) river the Kosi may be taken, with its bridge of fifteen spans of 200 feet. In diagrams illustrating the principles of the guide-bank system the bridge is invariably shown in the middle of a tangent, and, as far as the matter has been considered, the length of the guide-bank has been related to its effect in protecting an approach-bank in this position ; that is, on the line of the bridge. A reference, however, to *Fig. 5* will show that the right approach to the Kosi bridge is inclined at an angle of about 54 degrees to the line of the bridge, and that the right guide-bank, of a length of 1·33 L, has not been sufficient to protect it from the embayment at the back of the guide-bank, a retirement of the approach-line having been necessitated in order to avoid a break in communications. If, however, it had been possible to construct a right guide-bank of length 2·33 L, as shown by the heavy dotted lines, the embayment would have taken the position shown by the dotted line between the village of Madrauni and the head of the extended guide-bank, other conditions being equal, and the retirement of the main line to an alignment 4,800 feet distant would have

been avoided. Retirement of a line which has been carefully located usually leads to difficulties, and in this case the retired line was unexpectedly

Fig. 5.



River Kosi between Katareah and Kursela, November, 1936.

threatened by the encroachment of a bend of the Ganges, necessitating a switch, which was fortunately possible, back to the original alignment

near the bridge. It may be mentioned here as a matter of interest that this bend of the Ganges advanced upon the Kosi bridge in the first year (from 1897 to 1898) a distance of 2,500 feet, and in the four following years at the rate of 1,000 feet a year, after which a cut-off occurred in the Ganges bend, leaving a fairly active creek or spill in the position shown in *Fig. 5.*

The above remarks, however, are incidental, and the Kosi bridge has been cited here as an example of a bridge in a *khadir* with no banks. The Kosi issues from the foothills of the Himalayas at a distance of about 100 miles north of the bridge, and the fall of the river in this distance is about 200 feet. It may be taken that the fall at the bridge is about 1 foot in a mile. The river at its emergence from the gorge has, by means of spilling from its position on the alluvial cone, made channels for itself both east and west of its present position. In this way, at a few miles north of the bridge, a distance of 56 miles separates the most westerly from the most easterly channels to be found on the maps of the survey of India. It would therefore seem to follow that no railway-bridge over such a river could be made permanent by stone-pitched flanking guide-banks, whatever the ratio of their length to the width of the waterway might be. In his Technical Paper [1] Mr. A. W. C. Addis states that at a distance of 3 miles above the bridge the river moved laterally from 12 to 14 miles further west in the 7 years between 1896, when the surveys for the bridge were begun, and 1903, when his Paper was written. Instead, however, of breaching the railway at this distance from the bridge it came into a large river, marked Ghugri on the map, which parallels the railway and acts as a catchwater drain. The Kosi at the point of junction turned a right angle and flowed down to the bridge on a west-to-east course, now modified near the bridge to the course of 1936 shown in *Fig. 5.* The tongue of country on which the western railway-approach lies has never been penetrated by any of the rivers flowing down from the Himalayas, and it therefore seems probable that the tongue conceals *kunkur* deposits or possibily a reef of basalt of which the hills on the south of the Ganges are composed. The exploration of the east end of the tongue by means of borings would show whether there was any hard material which could be used to serve the purpose of deflecting the river to some better course through the bridge. The situation, however, now is that the railway-line from the westward has been saved fortuitously by the river Ghugri, that the approach-bank at the back of the right guide-bank is in danger, and, since old channels of the Kosi are found crossing the railway between the bridge and Katihar to the east, that nothing can save the railway from being breached and the bridge outflanked when the Kosi takes its next swing to the eastward.

The main deduction to be drawn from the examination of the conditions at the Kosi bridge is that the guide-bank system alone does not

[1] "The Bridge over the River Kosi." Technical Paper No. 138. Simla, 1903.

provide a permanent solution for the bridging of a river which is still engaged in building up the country through which it runs. Such a river may spill out of its self-raised channel, and, taking a new course, may breach the railway at a considerable distance from the bridge already built for it. In that case the only solution appears to be a new bridge to which the girders from the original bridge would be transferred. If, however, the river is subject only to local wandering in the vicinity of the bridge some permanence would possibly be attained by searching out and taking advantage of natural features, such as concealed *kunkur* deposits or hard clay beds, upstream of the training works, on which subsidiary works could be based ; such aids should by no means be rejected.

In addition, the example of the Kosi bridge, as it appears in *Fig. 5*, shows very clearly the disadvantage of the inclination of the western approach-bank to the bridge-alignment, and raises to a principle the desirability that the railway-line in the vicinity of class (3) rivers should be located as a tangent crossing at right angles the fall of the country, and of sufficient length to provide a site for the bridge, with approaches long enough to allow of embayment of the river at the back of the guide-bank. If, however, an approach-bank at an inclination to the bridge-alignment is unavoidable, the protection of the approach should be provided for by a suitable increase in the length of the adjacent guide-bank at the time of construction whilst such an extension is still possible.

The Hardinge Bridge over the Lower Ganges.

Although the final form of the training-works for the Hardinge bridge has not yet been determined, much may be learned from a study of the testing experiences through which the works have recently passed.

Breach in Right Guide-Bank.

On the 26th September, 1933, a breach occurred in the right guide-bank, for which no convincing explanation had been put forward until it was suggested by the late Mr. B. L. Harvey [1], M. Inst. C.E., that the breach probably started high up on the slope of the bank, at or just below water-level, and that it was caused by the suction, created by fast flow and especially by fast-moving eddies, drawing out silt or sand from between the pitching stones of the slope-covering. The protective covering in this case was 9 inches of stiff clay, 3 inches of quarry-refuse, and 3 feet 6 inches of pitching stone, and the material covered was a very fine sand with a large proportion of mica flakes and a high silt-content. The attacking force was a freshet of unusual violence, which interrupted a steady fall by a rapid rise of between 4 and 5 feet in river-level. The rise was accompanied in this

[1] "The Restoration of the Breach in the Right Guide Bank of the Hardinge Bridge." Journal Inst. C.E., vol. 4 (1936–37), p. 21 (November 1936).

part of the river by waves which are described by the observer as rushing through the gap in the guide-bank into the embayment at intervals of about 2 minutes and creating a sudden afflux of about 2 feet. It has been suggested by the present Author that the failure was due to the sand being dragged down and drawn out by these surge-waves passing along the face of the guide-bank, after the fashion, familiar to many, of the wave caused by a large ship passing through a relatively small channel. The weakness of the protective covering described above lies in the liability of alluvial clay, once dried, to dissolve in water, and in the possibility of weak spots in the covering of quarry-refuse, one weak spot in the circumstances of attack described being sufficient to cause a failure. As soon as it had been realized that alluvial clay was quite unreliable and that quarry-refuse varied in protective value, it had to be admitted that the construction was inadequate to meet the conditions of wave-wash produced by such a freshet, and that a fresh design was necessary.

Before coming to this conclusion the suggestion was further examined that the breach was caused by a deep-seated slip brought about by scour at the toe of the apron. The centre of the breach when first observed on the 20th September, 1933, was at chainage 20·5 of the right guide-bank. The latest information about the conditions at the guide-bank before the breach is contained in the contour-plan prepared in the last week of December, 1932. It may be assumed that these conditions would remain unchanged at least until the annual flood-rise beginning in June, 1933. The contour-plan shows that the toe of the apron, which had been reinforced by the addition of 400 cubic feet of pitching stone per foot run of bank deposited through water, occupied a position in plan 350 feet from the centre-line of the guide-bank from chainage 30 to chainage 15, where the toe appears to run back to 280 feet from the centre-line in 100 feet. The toe runs down very evenly from about 74 feet below low water at chainage 30 to 103 feet below low water at chainage 18, where an oval contour-line at R.L. 116 shows the deepest smooth-flow scour existing at any point along the guide-bank at that time. The contour-plan shows water's edge on the 26th December, 1932, and from its relation to contour R.L. 220 the water-level may be assumed without appreciable error to have been R.L. 225. On these data at chainage 20·5, afterwards to become the centre of the breach, the section from the foot of the permanent slope gives a fall of 1 in 7 for 90 feet to water's edge, followed by a steeper slope of 1 in 1·70 for 170 feet. At chainage 18, from the foot of the permanent slope there is a fall of 1 in 6·5 for 85 feet to the water's edge, followed by a steeper slope of 1 in 1·65 for 180 feet. It will be observed that the outer slope at both sections is steeper than the normal 1 in 2, but that in both sections there is a large margin of from 90 to 85 feet width, with the very flat mean slope of 1 in 6·5 or 1 in 7·0 to go down before any slip could reach the foot of the permanent slope. These being the conditions at the guide-bank under which the flood-rise in June would

take place, the course of events may be followed. The main current was described as coming down upon the curved end of the guide-bank and passing through spans 2 and 3, and this appears to preclude the possibility of any large eddy forming off the guide-bank. There is, however, no need of an eddy, and smooth-flow scour could, and no doubt did, take place. There was nothing in the direction of flow to press the main stream towards the guide-bank, and the maximum depth of scour would probably occur at some distance out from the toe of the apron and opposite chainage 20, which is the tangent-point of the curved head. Nevertheless, let it be supposed that the main stream did come in towards the guide-bank and scoured along the toe of the apron quite early in the annual flood-rise. There was plenty of margin for the apron to follow the scour down by small steps, but it may be supposed that this did not occur in order to assume the most dangerous condition. However, no breach occurred in either June or July, or in August when floods usually reach their greatest height, and the river was falling steadily by the middle of September and continued to do so up to the 25th September, when the river ceased to fall and the rapid rise of the freshet began. So far as can be ascertained by plotting the levels, the river fell to about R.L. 239·2 and the breach was observed after the river had risen to R.L. 240·2. As the breach had not occurred by some catastrophic slip on the 23rd September when river-level passed R.L. 240·2 on the fall, there would be no reason why a slip should take place at this level on the rise, and it was necessary to seek some other cause for the breach. The cause must lie in some differences between the flow of the river on the 23rd and the flow on the 26th September, and these are to be found in increased velocity of current accompanied by destructive surge-waves at 2-minute intervals on the latter date, as previously described. From these considerations the conclusion appears to be unavoidable that such waves could and did drag down and out through any weak spots in the protective covering the light unstable material of which the bank consisted, so as to allow the stone pitching to fall and to expose the core at water-level to waves and current, and that a complete breach rapidly followed.

Some corroboration of the belief that the breach was not caused by a deep-seated slip may be found in the circumstances of the catastrophic slip which took place in the following year on the 25th October, 1934, between chainage 5·5 and chainage 12·5 of the same guide-bank adjoining the stabilized breach. It is well-known that deep-seated slips usually take place with a falling river owing to the increasing weight of the part of the guide-bank left above water and the decrease in the horizontal pressure supporting any sand face there may be below the toe of the apron. In this case a flat tract of apron, 500 feet long and varying from 100 feet wide at chainage 5·5 rather abruptly to nothing at chainage 10·5, went down bodily within a few minutes, the slip extending deeply into the bank. The channel at the toe of the apron-slope had deepened during the flood-

season of 1934 from 49 feet below low-water level to something which must have been considerably more than the 94 feet below low-water level which was sounded after the slip had taken place. Nevertheless, it was not until the velocity of the current had diminished to 4·5 feet per second, having long ceased to scour, and the river had fallen to R.L. 234·00, at almost exactly half-flood level, that the slip occurred. In other words, in order to start the slip, it had been necessary for the river to fall 5 feet below the level at which it has been assumed that no such slip had taken place at the site of the breach, and there is, in the laying of the 400 cubic feet of pitching stone per foot run through water at the toe of the apron at the site of the breach previously alluded to, an additional reason why any deep-seated slip at that place should be delayed ; on the other hand, in the absence of such reinforcement at the toe and the additional weight in the reinforcement applied at the top of the undisturbed part of the apron, there are two reasons why the slip between chainage 5·5 and chainage 10·5 should have been accelerated.

There is no doubt that this slip of the 25th October 1934, was caused by the deepening of the channel along the toe of the apron, and the interest lies in the reason why two-thirds of the width of the apron went down in one slip instead of in a succession of small slips. The reason is to be found in the nature of the strata underlying the guide-bank. The borings taken in 1935, to explore the extent of the " clay patch," do not include any borings at the site of this slip or at the site of the adjoining breach, but those at the centre-line of the bridge on each side of pier 2 show that at 46 feet below low-water level the undisturbed clay-patch strata are met with. These strata consist of thin bands of clay, coarse sand, black sand and so forth, differing in every way from the recent deposits of fine white sand and silt found in the river-bed. They are traceable in borings taken in 1936 between chainages 2 and 6 of the guide-bank, and it is reasonable to believe that they extended upstream from chainage 0 to chainage 25, underlying the slip of 25th October, 1934, as well as the breach, since in the whole of this length the apron failed to respond to the depth of water alongside the apron to an extent diminishing from two-thirds of the width of the apron at chainage 5·5 to one-third of the width at chainage 24. In fact, at this guide-bank the apron failed to act as it might have been expected to do if the underlying material had consisted of pure sand.

Since this Paper was written, on the 1st April 1938, at the extreme end of the head of the right guide-bank, a slip took place " during a severe storm." The report of the slip, of which the cause is at present obscure, was followed by cross sections with borings which show that at the extreme end of the guide-bank the clay-patch strata in the form of the " coarse sand " stratum were met with at a depth of 104 feet below low-water level. Confirmation of the conclusion previously reached that the whole of the upstream portion of the right guide-bank rested on clay-patch strata, has thus unexpectedly been provided. These strata

have shown themselves highly resistant to smooth-flow scour, and their existence strengthens the view that the breach could not have been caused by scour at the toe of the guide-bank.

Further confirmation of the conclusion that the breach originated at water-level and was caused by surge-waves acting on defective soling, is to be found in the serious slips which had previously occurred in the slope-stone at the head of the same guide-bank, referred to below.

Occurrences at Head of Right Guide-Bank.

In his Report dated December, 1933–January, 1934, the Author remarked " The cross sections of the original pitching at chain 25 to chain 30 at the upstream end of the curved portion of the right guide-bank, dated 23rd January 1932, that is after the floods of 1931, show that the toe of the pitching had reached an average level of 60 feet below low water, that the average slope was 1 in 2·75 and it is stated that the pitching at the foot of the slope had slipped requiring repair. This is a remarkable divergence from the usual behaviour of pitching, it seems to show that the apron at the foot of the slope should be thicker." " Or the slipping of the stone at the foot of the slope might have been caused by wave action." Further examination of these sections confirms this diagnosis, especially regarding the insufficiency in the thickness (4 feet 6 inches) of the inner belt of the apron. At chainages 29 and 30 there is distinct gullying, the middle part of the inner belt having dropped 20 feet, giving the impression that the underlying sand had been washed out through the 4 feet 6 inches of pitching stone. At chainage 26 the inner belt had also dropped 20 feet, but in this case the whole slope is about 1 in 2·7, and it looks as if the outer part of the inner belt had dropped first and that the sand had washed out through the 3-foot thickness of stone resulting from it having been dropped to a 1 in 2 slope in the first instance. The head of this guide-bank has been subjected to persistent normal attack, and has suffered severely from slips in the slope between high-flood and low-water level due to insufficient soling in the " permanent " slope and insufficient thickness in the inner belt of the apron. This is confirmed by the circumstance that, owing to the flatness of the slope taken by the apron and the steepness of the slope above apron-level, it was found necessary to flatten the curve of the head of the guide-bank in order to allow this steepness to be reduced. It was during the flood at the time of the slip of the 3rd September, 1934, at chainages 27 to 30 that the edge of the main stream was observed to move out to 400 or 500 feet from the head of the guide-bank and to return to run hard against the bank several times during the day, and it was thought that this might be more destructive in effect than a steady stream.

Deepest Known Scour for the Purpose of Design.

It has been customary to design both bridge and training works on the greatest depth below low-water level which it has been possible to find by

soundings taken at a cutting bend, during and after high floods, and designated as "deepest known scour." If, however, the depth below low-water level has been taken at a cutting bend in contact with a soft bank, it is proposed to convert it, by the addition of 33 per cent., to the depth which it might be expected would be found if the bend were in contact with a hard bank; this depth is the minimum required for contact with a stone-pitched guide-bank, and it is this which will be referred to in future as the "deepest known scour" for the purpose of design. This is a smooth-flow figure, and it has been assumed that the training works will be so designed that no large eddies will be formed in the vicinity of bridge or training works. Eddy-scour has hitherto been considered much deeper and more dangerous than smooth flow, but it has been thought possible to avoid it. As to smooth-flow scour, experience shows that, owing to variation in the high-flood level reached and in other conditions, it is extremely unlikely that a maximum depth of scour would be obtained in any particular year, or oftener possibly than once in 33 years. It is therefore proposed to add to the deepest known scour a percentage for the pier-foundations and guide-banks, with a larger percentage for the heads of the guide-banks. In order to assist in the determination of the percentages to be taken for class C rivers, the depths found in different situations at the Hardinge bridge have been set out in Table I. The ease with which reliable soundings can be obtained in flood-time by means of the echo sounding-apparatus fitted in a fast launch, which has been available at the Hardinge bridge since 1934, may help to account for the discovery of the greater depths of some of the more recent soundings. With regard to the great depths found at the abandoned Sara protection-bank and at the upstream end of the Raita protection-bank, the flow in both cases is in a transitional state between smooth flow and eddy-producing flow, and the term "irregular flow" sufficiently describes it. At Sara the depth of 164 feet below low-water level is caused by the irregular water-line and the wreckage of the protection-bank, whilst at Raita the 192 feet below low-water level occurs exactly opposite chainage 40 at the extreme upstream end of the protection-bank, and is due to an embayment above the head, which at contour 225 is as much as 220 feet.

In deducing the appropriate percentage for class C rivers from the Table, no use can be made of the maximum smooth-flow scour of 124 feet off the curved head of the short right guide-bank because it is now known, from the reports on the slip which took place at the end of this guide-bank on the 1st April 1938, that the scour of 124 feet penetrated by 20 feet the resistant clay-patch strata. It had long been remarked that this was no great depth for a position which had been so heavily attacked. As this depth of scour did not represent the maximum depth of smooth-flow scour in deltaic Ganges sand, it became necessary to take into consideration the sounding of 144 feet below low-water level, obtained opposite chainage 22 of the Sara protection-bank under smooth-flow conditions. Although this

TABLE I.—SOUNDINGS AT THE HARDINGE BRIDGE TRAINING WORKS.

Date.	Soundings.					Remarks.
	Depths below low-water level: feet.			From water's edge.		
	Smooth flow.	Irregular flow.	Eddy.	Distance of sounding: feet.	Slope of bank.	
Jan. 1903 .	100	—	—	—	—	Scour-hole along clay bank at Sara. Depth taken as "deepest known scour."
1909–1910 .	—	—	—	—	—	No greater depth found than 100 feet.
1931 . .	—	—	170	—	—	Eddy 800 feet in diameter at Sara.
Feb. 1933 .	103	—	—	175	1 in 1·6	Right guide-bank, chainage 18.
Sept. 1934 .	—	—	187	600	1 in 3	On radial line at chainage 12·5 of mole at right guide-bank, about 600 feet out.
After Floods of 1935 .	144	—	—	290	1 in 2	Sara protection-bank, chainage 22.
Jan. 1936 .	—	—	134	—	—	260 feet downstream of pier 2, due to stone around the pier being connected with the apron-stone, and forming a spur.
Oct. 1936 .	124	—	—	325	1 in 2·2	Right guide-bank, chainage 26 (see p. 156).
March, 1937	—	164	—	750	—	Abandoned Sara protection-bank at chainage 25, and 750 feet from it.
Aug. 1937 .	—	192	—	540	1 in 2·5	Off the upstream end of Raita protection-bank, chainage 40.

sounding was the deepest found in the years 1934 and 1935, during which the bank was exposed to smooth-flow scour of the whole river, and although from the exposed position of the protection-bank this might be considered to be greater than would be experienced in the more sheltered position of the body and tail of the guide-banks, nevertheless provision for such depths in the latter position has been made by the addition of 45 per cent. to the "deepest known scour" for the case of class C rivers. The percentage to be taken for the heads of guide-banks of class C rivers has been derived as detailed below. It will be observed that the depths at the heads of the Hardinge Bridge guide-banks, tabulated under the heading of irregular flow, exceed those due to eddies, and that by providing for maximum irregular-flow scours, provision is made for maximum eddy scours. It is more difficult to avoid the accidental occurrence of irregular-flow scours than to avoid the formation of eddies, and the necessity of providing for the former

entails a radical change in the design of guide-banks for class C rivers. The change consists in the provision at the heads of such guide-banks of a percentage allowance sufficient to cover scour of all kinds. This is substantially complied with by the addition of 90 per cent. to the "deepest known scour." It will also be noticed that by the addition made above of 45 per cent. for the body and tail of guide-banks for class C rivers, the depths of eddy-scours, in the vicinity of the bridge, have been exceeded with the exception of the abnormal scour of 187 feet opposite chainage 12·5 of the mole at the right guide-bank, which was the result of the open breach and not the cause of it.

Under the section of the Paper dealing with the Curzon bridge (p. 141, *ante*) the percentages to be added to the "deepest known scour" for class A rivers have been deduced as 25 per cent. for the body and tail, and 50 per cent. for the head of the guide-bank. The percentages for class B rivers have been fixed by interpolation and they may now be repeated here for all three classes of rivers :—for body and tail of guide-bank : 25 per cent. for class A, 32 per cent. for class B, and 45 per cent. for class C rivers; for head of guide-bank : 50 per cent. for class A, 63 per cent. for class B, and 90 per cent. for class C rivers. These percentages have been entered in the explanatory notes on the diagram of the normal apron (referred to on p. 177, *post*).

Part II.—GENERAL PRINCIPLES.

Introduction.

As soon as it becomes apparent that a river on a line of railway cannot be bridged in the ordinary way owing to the unstable character of the course of the river, it is necessary to determine whether a permanent bridge is economically possible on the guide-bank system. This system in favourable conditions will ensure permanence but, in less favourable circumstances, can only be expected to prolong the life of the bridge. There are cases to which the guide-bank system cannot properly be applied. In order to assist the engineer who may be called upon to design a bridge on this system, some general principles may be formulated. As different rivers require different treatment the problem will be simplified by dividing them into three classes according to environment.

Class (1) includes rivers running between permanent banks in a valley or *khadir* excavated by the rivers themselves below the level of the surrounding country. If the valley is too wide to bridge from bank to bank a guide-bank bridge is indicated.

Class (2) includes rivers running in a valley or *khadir* of their own making below the level of the surrounding country. This differs from Class (1) only in the high banks not being permanent; the treatment, however, is the same, although modified according to circumstances if the river is going through a phase of eroding one or other of the soft high banks.

Class (3) includes rivers which are still engaged in building up the country through which they pass. The banks of such rivers are higher than the surrounding country and the rivers are free to wander to an unlimited distance in either direction. Such rivers require special treatment.

It is also necessary to divide the rivers into three classes according to size, that is to say, by magnitude of discharge :

Class A, rivers with a discharge of from 250,000 to 750,000 cusecs.

Class B, rivers with a discharge of from 750,000 to 1,500,000 cusecs.

Class C, rivers with a discharge of from 1,500,000 to 2,500,000 cusecs.

The rivers to which the system has been applied with varying degrees of success and from which these principles have been derived ranged in magnitude from 100,000 to 800,000 cusecs, with one example of about 2,000,000 cusecs. The classification by magnitude of discharge has been necessitated by the marked difference in the behaviour of the latter, in which the increase in magnitude has been accompanied (as it appears to be in the rivers of northern India) by an increase in the silt-content and a greater degree of fineness in the sand of the river-bed.

Location of Bridge.

Limitation of Flood-Fall of the River.

The flood-fall of the unobstructed river in the vicinity of the bridge-site should preferably not exceed 2·0 feet per mile.

River-Bed to be Easily Scourable.

In order that the river should be able to pass between the guide-banks and through the bridge without appreciable afflux, the bed of the river should be easily scourable throughout. To this end, before final selection of the site of the bridge, it should be ascertained by means of borings down to permissible depths of scour whether the bed of the river on the line of the bridge, as well as between and along the guide-banks, is free from obstructions such as partially-eroded beds of clay or *kunkur*, which may throw great stress on particular piers or parts of guide-banks.

Location of Bridge-Approaches.

Where possible the railway line should be located so that the bridge will fall in the middle part of a tangent. The safety of the approaches is as important as the safety of the bridge, and on the bridge-tangent the approaches are withdrawn as far as practicable from embayment of the river at the back of the guide-banks.

Determination of the Length of the Bridge.

To determine the length of the bridge it will be necessary to take two sets of observations, one in some straight reach, preferably that selected

for bridging, and one at any big bend of the river available. The observations will consist of soundings for cross sections and mean velocities for discharge, at different flood-levels. Each set of observations should consist of at least three determinations of discharge, one at about three-quarter flood, one at high flood, and one intermediate. The three-quarter flood observations at the bend are best taken on a falling river, as scour in the bend is then likely to be deepest. From each set of three determinations a maximum discharge for the ascertained maximum high-flood level can be obtained by producing the curve of the discharge-diagram. In order to determine the length of the bridge, the cross sections of the river bed should be plotted from the "bend" set of observations, using the ascertained maximum high-flood level as a datum. A compound cross section consisting of the lowest line of each part of these cross sections should be used in the following calculations. The low-water level should be marked, but the mean-velocity diagram should be plotted from the maximum high-flood datum, using the mean velocities observed at the high-flood discharge determination. A scale may be used which will bring the diagram below and clear of the compound cross section of the river-bed. If the high-flood level at which the discharge has been taken is reasonably near the ascertained maximum, the observed mean velocities may be applied to the cross-sectional areas between the maximum high-flood level and the compound cross section of the river-bed. It will be seen that on one side of the river the water will be shallow and on the other side deep. As for any surface-gradient the velocity increases with the depth, it will be found that by reducing the length of the water-line on the shallow side a large percentage of cross-sectional area of river can be excluded for a comparatively small percentage of discharge, and that this can be made up by a small increase in depth in the already deep part of the section. The deepening may be assumed to take place in one-third of the reduced width of the river. Proceeding by trial and error, and bearing in mind that the increase in depth will entail deeper foundations and provision for the deeper scour in the design of the guide-banks, a reasonably compact section for a reduced length of bridge will be obtained without too great an increase in depth of scour. The "straight" set of discharge observations should now be similarly used for the preparation of the compound cross section of the river and the velocity diagram, and a useful check will be obtained by trying the reduced length of bridge upon them. Whatever length of bridge—or rather, width of waterway—is decided upon, and however straight the reach of the river at the site of the bridge may be, the main stream in flood-time will be found to run as a rapid compact stream of a width varying from one-third to one-fourth of the waterway provided. It must, however, by no means be inferred from this characteristic of rivers with easily scourable beds that the width of the waterway should be further reduced. Owing, moreover, to the desirability that the two guide-banks should be symmetrical about the axis of

the bridge, that they should both be completed in the one working season, and that as small an amount of the training works as possible should have to be constructed in water, it is not unusual to find that a longer bridge is necessitated than would be required by the discharge-calculation.

Determination of the Depth of Pier-Foundations.

In order to determine the depth of foundations in sand it is usual to ascertain the greatest depth that can be found by sounding at cutting bends in the river in the vicinity of the bridge. The sounding is reduced to depth below low-water level as the most convenient datum, and is termed the "deepest known scour." The greatest depth will probably be found, not at the time of highest flood when the water is running straight down the flooded channel, but on a falling river at about three-quarter stage, when the channel takes on sinuosity and cutting bends appear. If the deepest known scour, on which the whole design of the bridge and training works is to be based, has been obtained at a cutting bend eroding a soft bank, it will be necessary to convert it, by the addition of 33 per cent., to the depth which it might be expected would be found if the bend were in contact with a hard bank. This is the minimum depth required for contact with a stone-pitched guide-bank, and it is this which will be referred to in future as "deepest known scour." Pier-foundations in the more easily scoured river-beds have never yet been deep enough, and it will be necessary to add to the depths found as above the percentages adopted for the guide-bank in the vicinity of the bridge in order to allow for increase in scour due to narrowing of the river and for the unlikelihood of arriving at the maximum depth of scour in cutting bends, by the observations of any one year. For minimum grip in the river-bed it will be sufficient to add, for class A rivers, 50 feet; for class B rivers, 55 feet; and for class C rivers, 65 feet.

For example, assuming a "deepest known scour" of 40 feet for class A, 70 feet for class B, and 100 feet for class C rivers, the depth of well-foundations in river sand would be :—

Class of river :	Class A.	Class B.	Class C.
Percentage addition for guide-bank in vicinity of bridge	25	32	45
Assumed depth of "deepest known scour": feet	40	70	100
Additions at percentages named : feet	10	22	45
Grip in river-bed : feet	50	55	65
Depths of foundations below low-water level in sand : feet	100	147	210

Protection of River-Bed around Piers.

As soon as the wells are sunk to full depth and founded they should be surrounded by an apron of pitching stone laid on the sand if the bed is dry, or dropped through water if the well is in the low-water channel. This apron or carpet of pitching stone is provided, not to hold up the pier by a mass of solid stone, but to prevent local scour due to the disturbance and increased velocity of the current around the upstream half of the pier and to the eddies caused around the downstream half. The stone goes down more rapidly and deeper under the eddies, which persist for some distance downstream of the pier, than it does under the influence of the increased velocity of the current. There is a slight tendency for the upstream stone to move downstream in dropping down into place, and, taking into consideration the probability at some time of obliquity of current, the best shape of the apron in plan is pear-shaped with the small end upstream.

Fig. 6.

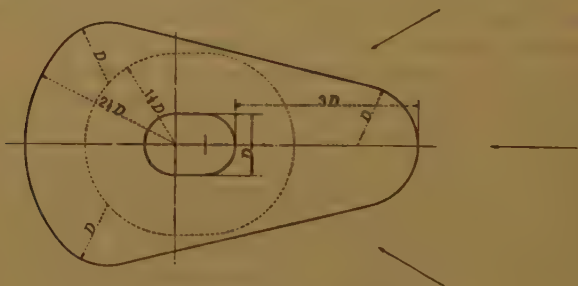


Diagram of Pier-Apron.

This disposition of the pitching is shown in *Fig. 6*, where the dotted line shows the plan often adopted and the full line the plan and relative area recommended to be covered. No matter to what depth the wells are sunk in the material found in these rivers, the stone apron, required to combat the local scour, cannot be dispensed with. In all cases in active rivers the expenditure on replenishing the stone round the piers has been enormous, partly owing to some misunderstanding of the purpose of the pitching, and partly to the difficulty experienced in verifying the position of the stone after the subsidence of the floods. After the flood season the local scour around the piers silts up, or the river-bed itself may silt up owing to lateral movement of the main stream, and in such cases it has been usual to locate the position of the stone apron by pricking with light steel rods. The art of pricking to great depths in sand, however, appears to have been lost, and the only sure way to avoid unnecessary expenditure is to put down borings to establish the position of the stone; if the apron originally has been sufficiently widespread downstream and the stone is found to be above

the depth of maximum permissible scour, it may be taken that no replenishment of stone is necessary at that pier. It has not been shown, in the rivers under consideration, that "one-man rock" pitching in mass lying on sand is ever bodily washed away downstream, nor that any advantage is gained by increasing the size of the pitching stone. On the contrary, especially around the downstream part of the pier, large concrete blocks and large boulders appear to sink deeper and more rapidly owing to the sand being sucked out more easily from the larger cavities by eddy action. In view of the necessity in all cases for the presence of pitching stone around the piers, and the desirability of leaving the river-bed in the middle of the span free to scour, it may be mentioned that in determining the length of the span to be used no objection should be taken to the cost of the steel span exceeding the cost of the pier by a reasonable amount. In placing pitching stone around piers it is of great importance to avoid any tendency to the formation of a weir on the line of the bridge.

Training Works.

The purpose of training works may be said to be twofold: (1) to guide the river through the bridge with as little obliquity as possible, and (2) to fend the river off the bridge-approaches in order to keep intact the line of communication.

The design of training works should be such as to avoid and to prevent the formation of eddies in such proximity with the training works themselves, or with the piers of the bridge, as to affect their security.

The training works will usually consist of a pair of guide-banks flanking the bridge. Each guide-bank has a minimum upstream length conditioning the flow of water through the bridge, and any greater length than the minimum is related to the length and inclination of the approach-bank to be protected.

Form in Plan of a Pair of Guide-Banks.

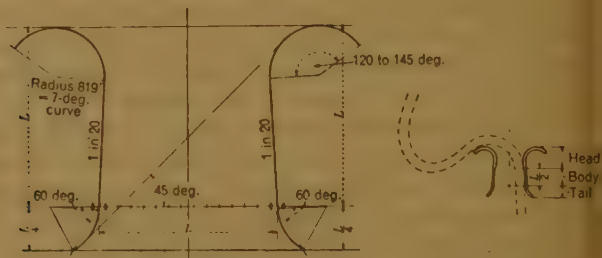
The Bell bund or guide-bank system of river-training consists primarily in the substitution of streamlined continuous stone-pitched guide-banks for the spur-system previously in use, and the consequent introduction of smooth flow in place of eddies. The principal feature of the bund and apron or guide-bank is the apron (Appendix, pp. 198 *et seq.*), which is laid in the dry as near low-water level as practicable, so that the work of distributing the pitching stone over the underwater slopes of the guide-bank may be done by the river itself.

The form in plan recommended by the Author for a pair of guide-banks flanking a railway bridge sited in the middle of a tangent (that is, with approaches on both sides in extension of the bridge centre-line), is shown in *Figs.* 7 (p. 164). Two diagrams have been prepared, of which one

(*Figs.* 7 (*a*)) is intended for general application at class A rivers, and the other (*Figs.* 7 (*b*)) for class C rivers. A diagram for class B rivers is

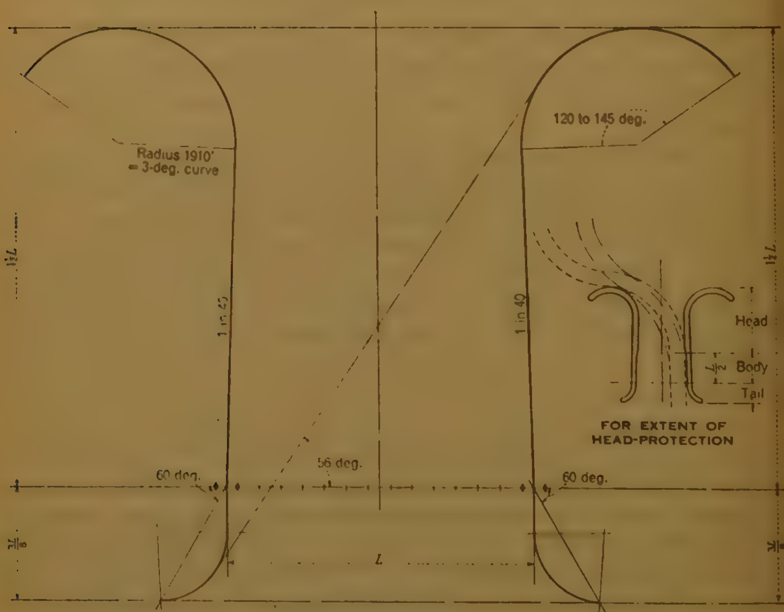
Figs. 7.

(*a*)



FOR CLASS A RIVERS.

(*b*)



FOR CLASS C RIVERS.

DIAGRAMS OF GUIDE-BANKS.

obtainable by interpolation. It is usual to take the length of the number of spans in the bridge minus one as the width of waterway, denoted by *L*, but to simplify the diagram this symbol is here applied to the distance between the guide-banks at the bridge.

Class A Rivers.—For class A rivers (*Figs. 7* (*a*)), the upstream length of the guide-bank is made equal to L, which is considered sufficient to ensure no undue obliquity of current at the piers. Subject to the assumption made with regard to ratio of cut-off (p. 143), this length L may also be assumed to be sufficient to protect the approaches of a bridge in the middle of a *khadir* between permanent banks not more than a distance of 7 L apart. If it is desired to protect a greater length of approaches the upstream length of the guide-banks should be proportionately increased. Similarly, if either of the bridge-approaches is inclined to the line of the bridge, the length of the adjacent guide-bank should be suitably increased.

If the general direction of the course of the river in approaching the guide-banks coincides with the axis of the bridge it is desirable that the guide-banks should be symmetrical in form. It is also desirable that they should converge to a throat upstream, each at an inclination of 1 in 20 as shown, in order to reduce obliquity of current at the piers and to hinder the formation of sandbanks in the throat. It has often been found convenient to make the straight part of the guide-banks at right angles to the bridge for the greater facility of construction on dry land, but it may be laid down as a definite rule that guide-banks of length equal to waterway should not diverge or be splayed upstream from the bridge, as all the advantages both of length and convergence as described above would in that way be lost. For reasons of economy the radius to be given to the curved head should not be greater than would be necessary to obviate danger from an eddy at point E in *Figs. 4* (*c*) (p. 148), where the main stream, coming out from the back of the guide-bank, would leave the curve. There is very little evidence obtainable on this important subject, but, from the absence of reports of damage from this cause in actual cases, a radius of 819 feet, or a 7-degree curve, is suggested for class A rivers. A 7-degree curve may be rather large for the smaller rivers, but 819 feet radius is suitable for broad-gauge ballast-train rolling stock, and this may become important in case of a severe attack by the river on the guide-bank head. The curves are shown subtending an angle of 145 degrees, but they need not be curved back further than required by the circumstances of the case. The circumstances differ with the width between permanent and semi-permanent banks in class (1) and class (2) rivers, and it may be mentioned that the subtended angle differs not only according to the physical conditions of the river but according to the alignment of the guide-banks (whether converging or splayed), and the criterion is that the curve need not be continued further than will ensure that the curve of the deepest anticipated embayment will remain tangential to the curve of the guide-bank head. The subtended angle will ordinarily be something between 120 and 145 degrees. A length downstream of $L/4$ as proposed by Bell appears to be suitable for class A rivers, and a construction which allows for a short extension downstream of the guide-bank straight followed by a curve is shown on the diagram.

It will be observed that *Figs.* 7 (*a*) include a diagram showing, by means of a possible course of the main stream, the position to which it is considered the additional thickness of covering and width of apron, proposed for the head of the guide-bank, should be extended. It will be seen that the protective covering of the head should be extended to a distance of ½ *L* from the bridge, where the change in the width of apron should be made gradually.

Class C Rivers.—The diagram (*Figs.* 7 (*b*)) for class C rivers is of limited application. It has never to the Author's knowledge been contemplated to bridge a class C river in a class (3) locality. The cost is too great for a purely temporary bridge. It is only where there are natural features, permanent or semi-permanent, limiting the oscillations of the river that such a bridge could be considered. The natural features must exist on both sides of the river and it is only downstream of them that a site can be selected. At such a site no embayment could occur at the back of the guide-bank, such as is shown in the small diagram in *Figs.* 7 (*a*), and the worst embayment to be provided for would be such as might allow the main stream to cross the head of the guide-bank at right angles, as indicated in the small diagram in *Figs.* 7 (*b*). In this diagram it will be seen that, in order to remove the attack on the left guide-bank from the immediate vicinity of the bridge, it has become necessary to increase the upstream length of the guide-bank from *L* to 1½ *L*. The course of the main stream, described as including an attack on the head of one guide-bank, an attack on the other guide-bank at the bridge-abutment, and an attack on the foundations of the adjacent piers, may be said to be the most dangerous attack to which existing guide-bank bridges can be exposed. It is believed that this triple attack can be avoided by making the upstream length of the guide-bank 1½ *L* as described above. On account of the greater length the inclination of the guide-banks has been reduced from 1 in 20 to 1 in 40. A length downstream of ⅗ *L* has been provided. The radius of the head, in view of the greater magnitude of class C rivers, has been increased from 819 feet to 1,910 feet, equivalent to a 3-degree curve.

The diagram (*Figs.* 7 (*b*)) for class C rivers has a very restricted application. It might have been used as the first construction at the site of the Hardinge bridge, with the Raita and Sara protection-banks made permanent as outlying training-works. Instead of full-length guide-banks or some such pattern, however, short flanking guide-banks were provided, trusting to the fair alignment of the Sara bank and the reputation for resistance to scour of the Sara clay. It is idle to speculate on what the result might have been if the clay bank immediately above Sara had been of the same quality as the shelf of clay at Sara, or if when the bank above Sara began to be eroded, smooth-flow could have been maintained. The guide-bank system is based on the ensurance of smooth flow by the design of the training works and on the strict observance of the principle in all works during construction and maintenance. Experience at the Hardinge

bridge shows that accidental circumstances may make this difficult or impossible, and although the principle should be kept continually in mind, there is need for some concession in the direction of providing protection for somewhat greater depths than those due to maximum smooth-flow scour.

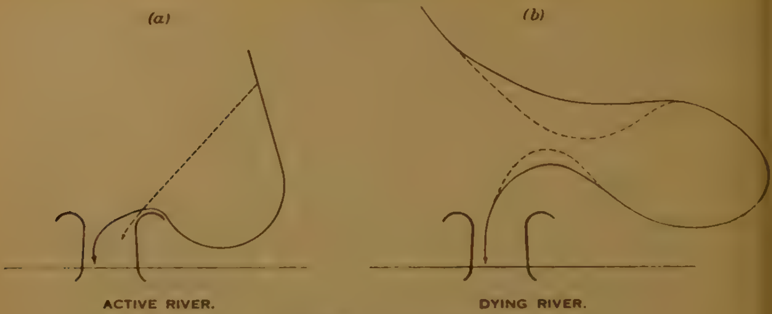
In considering the extent to which the application of the diagram (*Figs. 7* (*b*)) to class C rivers has been limited, it should be remembered that the Ganges in the delta is the only river in class C of which there is any recorded experience, and the respect with which the problem of bridging it was originally approached has been increased by the accidents which befell the guide-bank. On the other hand, a great deal more is known about the character of the sand of the river-bed and banks, and if the accidents to the guide-bank and the phenomenal depths of irregular scour attained at Raita and Sara have been rightly ascribed to the character of the sand, it may well be that higher up the rivers sites may come under consideration where the fine silt may be absent and the proportion of mica flakes reduced. It would then become a matter of judgement whether any of the requirements specified could be relaxed. One over-ruling requirement which has not yet been specifically mentioned will always remain in connexion with these long guide-banks: namely, that the site must be such and the river in such a position that there is no possibility of any change during at least two working seasons, for guide-banks can only be built on dry land.

Cut-Offs.

As it is the principal function of the heads of guide-banks to induce cut-offs, and by this means to bring back between the guide-banks the river which would otherwise have breached the approach-line and short-circuited the bridge, a few words on this neglected subject will not be out of place here. In rivers having their origin in the Himalayas there may be found, almost side by side, the existing active river carrying all the flood-water from the interior valleys and the old bed of the same river still kept alive by the drainage from the outer slopes of the foot-hills. The active river will be found to run in long easy curves and the dead or dying river in curves of increasing sinuosity, and from these characteristics two forms of cut-off appear to arise as illustrated in *Figs. 8* (p. 168).

The active river embaying at the back of a guide-bank will cut-off at a low ratio of bend to chord because flood-water will submerge the sandbank formed on the convex side of the bend and, by taking the short course along the chord, will form a channel before the growth of grass and bushes makes this too difficult. The dying river, on the other hand, will increase its curvature by eroding the concave curves, and, if floods no longer submerge the interior sandbank, the curvature will continue to increase until either the velocity of the stream is so reduced that the banks cease to be eroded, or until the loops cut into one another and a cut-off occurs

Figs. 8.

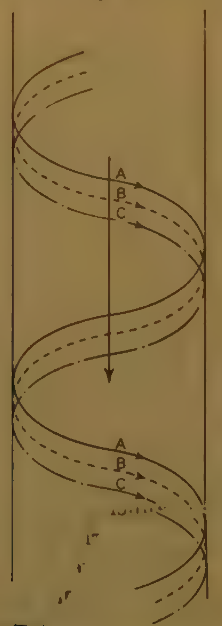


TYPES OF CUT-OFF.

by bend-erosion. Such cases are, however, few and less important, an[d] it is to the dominating part played by cut-offs in active rivers, in bringin[g] about the success of the guide-bank system, that it is desired to cal[l] attention.

It is reasonable to suppose, and it appears to be borne out by observa[-]tion, that the serpentine course of an active river running in sand betwee[n] fixed banks would travel continuously (though slowly) down country, a[s] suggested in *Fig. 9*, without the occurrence of any cut-offs; but that i[t]

Fig. 9.



SUGGESTED TRAVEL DOWN COUNTRY OF THE SERPENTINE COURSE OF A RIVER.

this movement were checked at any point by natural features such as *kunkur* or clay beds formed in the alluvium, the bend of the river would crowd down upon the obstruction until the ratio of length of bend to length of chord had increased to such a degree that in one or more flood seasons a cut-off channel would be established on the line of the chord. When an alluvial river has been bridged and provided with flanking guide-banks the head of one or other of the guide-banks has the same effect on the regime of the river as has been attributed to natural features, and the resulting cut-off brings the river back to the bridge on a more direct course.

It would seem that every alluvial river has its own particular ratio of length of bend to length of chord at which it may be expected to cut-off, and that the ratio would vary according to the characteristics of the river at the site of the bridge, such as magnitude of river, height of flood-rise, surface-fall, material of bed and its suitability for growth of protective grass and bushes, and so forth. The Author's contribution to this interesting subject is a belief that the Chenab at Sher Shah during the construction of the bridge looped in at the back of the right guide-bank and cut-off in the same year, and the information that the great Golbathan bend of the Lower Ganges just below the Hardinge bridge began to cut-off in 1911 at a ratio of 1·75 and that the cut-off became completely established in 1915 at approximately the same figure, the river having taken about 5 years to excavate the channel.

Although the bend and cut-off is the principal means by which the river recovers position to pass through a guide-bank bridge, it is not the only one, and the case of the Elgin bridge over the river Gogra may be instanced as one where the river embayed at the back of the right guide-bank, leaving behind it a rising sandbank blocking the direct entrance between the guide-banks. This high sandbank became covered with grass, reeds, bushes, and young trees, forming what appeared to be an insuperable barrier. The river, however, in the course of a westward movement above the bridge, encountered a bed of *kunkur* which deflected the main stream back towards the axis of the bridge. The deflected stream cut into the sandbank by lateral erosion and thus returned, although only temporarily, to a straight course through the bridge.

Influence of Silt-Content and Fineness of Sand on the Design of a Guide-Bank Bridge.

The content of silt in more or less colloidal form and the degree of fineness in the sand of the river-bed affect the design of bridge and training-works in two ways. The degree of ease with which the sand can be washed out through the voids in a guide-bank covering of pitching stone affects the thickness of the covering to be applied, whilst the ease with which the

river-bed can be scoured affects the width of the guide-bank apron and the depth of the pier-foundations.

With few exceptions the rivers of northern India have their origin in the Himalayas, and the sand of these rivers contains a large percentage of mica flakes. The sand from rivers on leaving the hills would be classified as medium, further down as fine, and on nearing the sea as very fine. A fine sand might be half fine quartz and half mica flakes. The mica flakes, although of higher specific gravity than the quartz, are more easily moved by currents. There would seem to be little difference in the behaviour of fine and very fine sand with similar proportions of these ingredients when used for the purpose of guide-banks, and it is suggested that a grading of these Himalayan sands by content of silt would be more useful. Tests of the sand which formed the Hardinge Bridge guide-bank which was breached during a freshet probably averaged 33 per cent. of fine granular quartz, 33 per cent. of small mica flakes, and 33 per cent. of silt, but it has not been established whether the original movement in the covering of pitching stone resulted from the silt being washed out of the sand or from the surge-wave action drawing out the mixture of sand and silt through the interstices of the covering. The determination of the silt-content is easily made by shaking up the sand in a graduated glass tube with a sufficiency of water and allowing the mixture to settle. The quartz sand and mica flakes come down together without loss of time, and the silt follows later.

With regard to the washing out of sand through the voids in protective coverings, very little is known about the behaviour of sand of different degrees of fineness and sand containing different percentages of silt when acted upon, through different types of slope-covering or through varying thicknesses of pitching stone below the level of the slope-covering by water of rivers of different degrees of magnitude flowing with different velocities under different conditions of smooth flow, concussion (as of waves) direction of flow, periodic change of direction of flow and change of pressure and with varying percentages of sand and silt in suspension at different depths of water in the main stream. An investigation of this subject would give results of the greatest usefulness, as there is little recorded experience to assist the designer.

With regard to the ease with which the above-mentioned three classes of river-bed sand can be scoured, there would seem to be no doubt that other things being equal, the finer the sand the greater would be the depth of scour, and that in the class termed "very fine, including silt" the depth of scour would increase rapidly with the proportion of silt. Conditions which are likely to affect this simple generalization are the degree of compression to which the sand has been subjected, and whether its mobility has been affected by infiltration of lime. Compression of very fine sand containing silt in colloidal form would tend to produce a structure which would stand with a vertical face under water without offering any increased

resistance to scour. The infiltration of lime in sand under pressure would cause a noticeable resistance to scour, however slight the infiltration might be. This infiltration, to which attention has been drawn in the previous part of this Paper, may be more common than might be expected from its rare appearance on the surface, as may be inferred from the fact that the well put down for the power-house, on the same sandbank as the right guide-bank during the construction of the Hardinge bridge, produced water of such hardness as to necessitate a water-softening plant, although the Ganges water circulating through the sandbank with the rise and fall of the river was quite remarkably soft. These considerations emphasize the need of the borings called for under the section of the Paper dealing with the location of the bridge (p. 159), and that they should be core borings taken at intervals along the centre of the aprons of the guide-banks proposed.

Considerations Affecting Slope-Covering and Apron for Guide-Banks Formed of Sand.

For the design of guide-bank slope-covering and apron, reference may be made to Figs. 10, Plate 1, and to Table II (p. 175).

In general it may be said that a fully-developed guide-bank consists of two parts: (1) the upper part above the original level of the river-bank or bed, consisting of an artificial bank constructed of sand excavated from the apron borrow-pit on the river-side; and (2) the lower part, consisting of the natural river-bank or bed. The upper part is constructed with a face slope of 1 in 2, and if the apron drops as anticipated, the face of the lower part of the guide-bank becomes a continuation of the 1-in-2 face of the upper part in the guide-banks hitherto constructed. There are thus two parts to be protected—that is to say, to be furnished with some covering, as of pitching stone, which will prevent erosion of these sand surfaces by river-action. To the upper part, which is necessarily between high-flood level and low-water level, the covering can be applied in the dry during the low-water season; it may take the form of a soling consisting of some impervious bed, as of clay, or porous bed, as of stone ballast, covered by a layer of pitching stone of definite thickness, and if this protective covering is not to be disturbed some provision must be made to ensure the permanence of the slope. The provision proposed is a width of apron at the foot of the slope, described as the berm, additional to the apron which is intended to pitch the lower slope.

As the continuation of the 1-in-2 slope will be mainly below low-water level and the covering will necessarily be applied to the slope by the action of the river itself, and since the only material which has proved to be suitable is pitching stone of a size sometimes described as "one-man rock," it follows that the design of the covering resolves itself into the determination of the thickness of the layer of pitching stone for the under-

water slope, or, more strictly, for the under-apron slope of the guide-bank in question. The thickness required is that which will suffice to prevent the washing out of the sand through the interstices of the stone by the action of the river. Having designed the covering for the permanent slope and determined the thickness of the stone covering for the under-apron slope, the amount of stone required for the latter purpose is known, and all that remains to be done is to arrange its disposition in the apron so as to provide the greatest measure of protection where it is most required.

It will be observed that the design of slope-covering and apron is intended for guide-banks consisting of sand only, both above and below apron-level, and it may be mentioned here that, where guide-banks are sited to include bands of clay or other material below apron-level, special designs adapted to the circumstances of the case become necessary.

Theory of Apron.

Before proceeding to the design of the apron, some consideration may be given to the method by which the river transforms the apron of pitching stone, laid in the dry, into the stone revetment of a sub-aqueous slope. From its position on the bank of an alluvial river the apron will usually be laid on the sand of the river-bed, and pure sand to the depth of the pier-foundations is the proper material for the purpose. The sands with which the guide-bank system of river-training is mainly concerned have been divided into three classes as medium, fine, and very fine with admixture of silt. In all three cases when the rising river causes scour for the first time along the outer edge of an apron, the stone should proceed to its final position by means of small slips in the sand below the apron. The size of the slip depends on the extent to which the sand has been compacted by time or rendered capable of standing with a vertical face under water. It has been stated that practically vertical cliffs of pure sand, standing more than 30 feet high, were to be found under water in the Lower Ganges. If it may be assumed that the width of the strip of apron to come down would be in the vicinity of half the depth of the vertical face, the slip would be a small one for an apron in the Lower Ganges. It would then take not less than four such slips for the toe stone to reach its final position. Each of these slips in turn would lead to a series of slips advancing towards the inner part of the apron, the depth of the face of sand exposed at each of these subsidiary slips being steadily reduced. What appears to take place, when the first slip occurs at the outer edge of such an apron, is that the weight of the strip of apron assists to shear the compacted sand in a plane, vertical at the inner edge of the strip but curving out to the vertical face of sand at the depth of 30 feet. After the strip of apron has dropped a short distance, the slip comes to a stop resting on the toe of the slip which has been pushed out into the channel; the toe of now loose sand is

rapidly scoured away and at the further movement of the slip the compacted structure of the sand breaks up and the apron-stone drops nearly vertically through the dissolving sand, spreading slightly forward and protecting a remnant of the slip. This leaves an exposed vertical face of perhaps only 15 feet for the next slip of this series. There is no doubt that, with reasonable variations, horizontal aprons of pitching stone are transformed by the action of the river alone, in the manner described above, into the evenly distributed stone revetment of a subaqueous slope of 1 in 2, the toe of the apron dropping down an inclination of 2 to 1, within reasonable limits of accuracy.

At the Hardinge bridge, however, on the straight portion of the right guide-bank, where so many untoward occurrences took place, a part of the apron behaved in an entirely different manner. On the 25th October, 1934, as previously related, a tract of apron 500 feet long of a maximum width of 100 feet went down bodily within a few minutes. The slip extended into the bank so that the service-track was unsupported. The outer belt of the apron had gone down normally and, so far as can be ascertained, the channel at the toe of the apron-stone had deepened by about 50 feet without any previous effect on the apron. The unusual behaviour of this part of the apron is attributed to the survival below it of an older formation of sand strata interspersed with thin clay beds which appeared to have escaped erosion when the river recently passed over it. Such cases are rare and, as they can be guarded against by the taking of borings before siting the guide-bank, they need not impair confidence in the apron system.

As the opinion appears to be widely held that pitching stone will slip or slide down a sand slope of 1 in 2 on the slope of the bank above the apron and in the apron itself, this opportunity is taken of saying there is no danger of pitching stone slipping or sliding or launching down a sand slope of 1 in 2.

Design of Protective Covering of Guide-Bank.

The design, as may be seen in the Figs. 10 (*b*), Plate 1, includes the covering for the prospective slope (from which is obtained the quantity of pitching stone required by the apron), the disposition of the stone in the apron, the berm which is an extension of the apron, and the covering for the permanent slope.

Critical Analysis of Tapered-Apron Design.

In the tapered-apron diagram for slope and apron-pitching, Figs. 10 (*a*), Plate 1, which has been in general use, the thickness of the apron, where it meets the slope, is made equal to the thickness of the slope-stone. The thickness of the slope-stone is set out in the form of a Table based on a

classification of sand in five classes from very coarse to very fine, together with a classification of rivers in five classes from those having a fall of 3 inches per mile to those having a fall of 24 inches per mile. For rivers of the same magnitude the fall per mile would give an indication of the velocity of current to be expected, but it appears to have been overlooked that a large river with a low fall per mile may have as high a velocity as a smaller river with a greater fall per mile. The result is that the thicknesses of slope-stone given in the Table are insufficient as a basis for design of the aprons for large rivers. This is particularly unfortunate because the tapered-apron diagram for design of apron shows a section thick at the outer edge but tapering to the thickness of the slope-stone at the inner edge where it meets the slope, and if this part of the apron drops to the prospective 1-in-2 slope the insufficiency becomes more pronounced at a position where no sand-retaining soling has hitherto been considered practicable. If this inner part of the apron is laid at about half-flood stage, as will usually be the case at the larger rivers, this insufficiency of covering falls at the very spot on the section where the maximum thickness of covering is required. An example will make the insufficiency quite clear. The thickness of the slope-stone T for the Hardinge bridge guide-bank from this Table becomes 3 feet 6 inches, and the thickness of the apron at its junction with the slope is the same. If the inner strip, 10 feet wide, of this apron drops to the prospective slope of 1 in 2, the thickness of the stone covering lying directly on the sand will be 2 feet 6 inches, although the contiguous slope covering of 3 feet 6 inches thickness may be inferred from the text to require some kind of soling. The apron as actually laid at this bridge, founded on the tapering principle but tapering in steps, would have provided an equally inadequate thickness of 3 feet if the prospective slope had at any time been fully developed.

The inadequacy of the thickness of that part of the apron adjoining the permanent slope, moreover, is concealed in the tapered-apron diagram, which shows, throughout the whole of the developed slope, the average thickness of 1¼ times the slope-thickness, as if this were a result which might be expected to occur; whereas, on the assumption made in the diagram that the stone would descend along an inclination of 2 to 1, the greatest thickness on reaching its final position would be two-thirds of the thickness at the position in the apron from which it started, and for the third of the developed slope adjacent to the permanent slope the thickness would be less than T, varying from $\frac{2}{3}$ T to T.

The design of apron in common use having been shown to be defective and found to be quite unsuitable for large rivers, the Author ventures to put forward a diagram of guide-bank section with apron (Figs. 10 (*b*), Plate 1), and a Table of thicknesses of protective covering (Table II), in the preparation of which advantage has been taken of the experience gained at the Hardinge bridge. The diagram hitherto in use has been reproduced as Figs. 10 (*a*), Plate 1, for ease of comparison.

TABLE II.—THICKNESS OF PITCHING STONE AND SOLING FOR PERMANENT SLOPE AND THICKNESS OF PITCHING STONE FOR PROSPECTIVE SLOPE, AND FOR BERM AND APRON REQUIRED AT HEAD OF GUIDE-BANK AND BODY AND TAIL OF GUIDE-BANK, RESPECTIVELY.

The Thicknesses in this Table are intended to be Applicable without Change to the Rivers of Northern India.

Rivers:	Class A: discharge 250,000 to 750,000 cusecs.		Class B: discharge 750,000 to 1,500,000 cusecs.		Class C: discharge 1,500,000 to 2,500,000 cusecs.		Remarks.
Part of guide-bank:	Head.	Body and tail.	Head.	Body and tail.	Head.	Body and tail.	
Permanent slope—							(1)
Pitching stone, T	3′ 6″	3′ 6″	3′ 6″	3′ 6″	3′ 6″	3′ 6″	To be hand-set.
Soling ballast	7″	7″	8″	8″	9″	9″	Ballast broken to pass $2\frac{1}{2}$″ ring.
Thickness of covering, T_2	4′ 1″	4′ 1″	4′ 2″	4′ 2″	4′ 3″	4′ 3″	
Berm—							
Pitching stone, $1\cdot5\, T_1$	8′ 3″	7′ 0″	9′ 3″	8′ 0″	10′ 6″	9′ 3″	(2)
Prospective slope—							
Pitching stone	3·50′	3·50′	3·50′	3·50′	3·50′	3·50′	
Add—							
for absence of soling 33 per cent.	1·17′	1·17′	1·17′	1·17′	1·17′	1·17′	
for magnitude of discharge graduated, 22 per cent.	—	—	0·385′	0·385′	0·77′	0·77′	
for high silt-content graduated, 22 per cent.	—	—	0·385′	0·385′	0·77′	0·77′	Silt-content 33 per cent. in Hardinge bridge sand.
for head, 22 per cent.	0·77′	—	0·77′	—	0·77′	—	
	5·44′	4·67′	6·21′	5·44′	6·98′	6·21′	
Thickness of pitching stone, T_1	5′ 5″	4′ 8″	6′ 2″	5′ 5″	7′ 0″	6′ 3″	Designed for 1-in-2 slope.
Apron-thickness, $1\cdot5\, T_1$.	8′ 3″	7′ 0″	9′ 3″	8′ 0″	10′ 6″	9′ 3″	(3)

(1) Where true clay or insoluble *kunkur* earth is available, an outer covering, 2 feet thick, of this material may be substituted for the sand within the profile of the bank and for a width of 10 feet below the bottom of the berm, and a layer of stone ballast should then suffice for soling. Alluvial clay or other soluble material should not be used on the river side to cover the sand core of the bank.

(2) The soling on the permanent slope should be extended under the berm for a width of 10 feet, taking the place of an equal volume of pitching stone.

(3) Even if the circumstances should permit the apron on a curve to be constructed of full width and thickness, it would nevertheless be necessary to provide an additional quantity of pitching stone to allow for the fanning out of the apron in falling forward and down to its final position. The additional stone may be found by calculation and laid, as an addition to the thickness given in the Table, tapering from zero at the inner edge to the calculated thickness at the outer edge of the apron.

Classification of Rivers by Magnitude of Discharge.

The rivers are classified by magnitude of discharge into three classes, A, B, and C. In the circumstances of northern India the classification by discharge appears to be likely to follow classification by silt-content of sand; for example, the Ganges at its exit from the hills is a comparatively small river of which the discharge is added to, at intervals, by affluents from the Himalayas and by two other great rivers from central India, so that its discharge is continually increasing until the delta is reached. It begins in class A, passes through class B and flows through the delta in class C. The sand of the Ganges and of its affluents is classified as medium on leaving the hills, as fine in its middle courses and as very fine in the Lower Ganges where it passes through the delta. In the delta at the Hardinge bridge the sand which proved to be so unreliable at the right guide-bank was found to contain 33 per cent. of silt. The alluvial soil of the delta is notorious for instability, as shown by sinkage under the weight of banks thrown up upon it and by slips in the banks themselves, and the silt-content may be taken to indicate the measure of instability. It is probable that the Brahmaputra (which shares the Bengal delta with the Ganges), and the Indus, the other two great rivers of northern India, have somewhat analogous courses and characteristics.

The classification of rivers by magnitude of discharge for the purpose of the design of guide-banks follows from an examination of the problem of maintenance. The forces to be considered are those of attack opposed by those of defence. If the Ganges may be taken as typical of the three or four big rivers of northern India, it will be observed that if the power of the attack increases with the magnitude of discharge, whilst the resistance to movement of the sand of the river-bed decreases, a point might easily be reached at which it would be impossible to maintain a guide-bank at any cost. The conditions under which the sand is required to exert resistance to movement are on the face of the guide-bank under the protective covering, and in the bed of the main stream of the river at the toe of the fully-developed apron. That the power of the attack increases with the discharge follows from the observation that the maximum velocity of current of the smaller discharge, at the point where the river first becomes bridgeable on guide-bank principles, does not greatly differ from the maximum velocity of the river at its greatest discharge in the delta, although the rate of fall in the delta is much less. The destructive power of the river at any moment is therefore at least in proportion to its magnitude. Moreover, whereas high flood, where the river is first bridgeable on guide-bank principles, might last for 2 or 3 days, it would last 2 or 3 weeks, or it might even be said 2 or 3 months, in the delta, where the destructive power consequently would be incalculably greater. In addition, the surface-effects of the larger expanse of water would be greater in the larger river.

Revision of Design of Protective Covering.

The defects in the tapered apron disclosed by the analysis on p. 173, and confirmed under trial in the curved head of the Hardinge bridge guide-bank, as described on p. 155, call for an early revision of the design. The weakness discussed on pp. 151–155 in the slope-covering, which failed under severe conditions and with disastrous consequences at or near the junction of the curve with the straight of the same guide-bank, also demands attention. The revision is based upon the considerations discussed on pp. 169 and 171.

The changes introduced in the new or normal-apron diagram (Figs. 10 (*b*), Plate 1) concern both the width and the thickness of the apron. The width of the apron, being 1·5 D, is governed by the depth of water arrived at by the addition of allowances, and it will be seen from the notes on the diagram that if the deepest smooth-flow scour below low-water level ascertained for the original project has been found where the bend is cutting a soft bank, an addition of 33 per cent. is to be made to bring the depth up to what it is assumed would have been the depth if the bend-scour had been found in contact with a hard bank, that being the minimum depth to be expected for contact with a stone-pitched guide-bank. To this basic depth D_1 (to which the term "deepest known scour" has been applied) is to be added the appropriate percentage allowance D_2, detailed for each of the three classes of river on p. 158 and repeated here :—

Class of river:	Class A.	Class B.	Class C.
Percentage addition to "deepest known scour" to be made for:—			
body and tail of guide-bank	25	32	45
head of guide-bank	50	63	90

These percentage-additions, D_2, are for contingencies such as the unlikelihood of finding the absolutely deepest scour in the course of a single flood-season, and include for the reduction in the width of waterway in the case of the body and tail of the guide-bank, and include for exposed position in the case of the head of the guide-bank. The height above low-water level at which the apron will actually be laid, D_3, is also to be included in D, the depth for calculation.

The principal change in the apron is the thickening of the inner part at the foot of the slope and reverting from the tapered form to an apron of uniform thickness, hereafter referred to under the name of the "normal apron." In class A rivers, where the quantity of stone remains about the same, the effect is naturally to reduce the thickness at the outer edge, but in class B and class C rivers, owing to the greater quantity of stone employed, the thickness at the outer part of the apron is not greatly altered.

The apron for the head is also made thicker than for the rest of the guide-bank.

Before proceeding to the detailed design of the thickness of the covering for each part of the guide-bank section, it remains to determine which part of the section requires the greatest measure of protection.

It will have been recognized that high velocity, aided, as high velocity usually is, by turbulence (including moving eddies, periodic changes in direction of flow, pulsations and the like), would be a potent agent in washing out sand through the voids in a covering of pitching stone, and that this means of destruction would be most effective in the upper half of the depth of water at high flood. To the high-velocity means of destruction, between high-flood and low-water level, must be added that due to surface-disturbance. The greatest displacement of underlying sand appears to arise from surge-waves, due to current-velocity, which waves, in passing down the river, drag down and draw out the sand from the face of the guide-bank. Other surface-disturbances may be waves due to winds of cyclonic character, "nor'-westers," and wave-wash from river steamers. It follows from these considerations that the face of the guide-bank between high-flood and low-water level requires the greatest degree of protection; that is to say, in terms of pitching stone, the greatest thickness of covering. It is a general principle that the apron should be laid at as low a level as circumstances permit, and in class A rivers the level of the bottom of the apron may approximate to quarter-flood level and in class C rivers to half-flood level. The composite protection of the permanent slope and the protection by pitching stone alone of the slope below the apron should therefore be of equal value. It may be repeated here that the causes which move grains of sand under protective coverings, such as high velocity of current or surface-waves, produce an effect proportionate to the duration of the high-velocity current or to the persistence of the wave, and that this appears to supply the principal reason why the larger rivers require the thicker guide-bank coverings.

The thickness of the covering of pitching stone found to be appropriate for the slope above low-water level would be continued without question down to half the depth of smooth-flow scour below high-flood level, but it would seem that in descending through the lower half of this depth safety would be secured with a diminishing thickness of stone, for two reasons: namely, the reduction in velocity of the current, and the increase in the turbidity of the water, tending to reduction of the amount of sand and silt which the moving water would pick up. Nevertheless, it is not intended to make any reduction in the thickness of cover provided.

Permanent Slope Covering.

Referring to Table II (p. 175) and Figs. 10 (*b*), Plate 1, it will be seen that for the permanent slope it is proposed that, where true clay or insoluble *kunkur* earth is available, an outer covering 2 feet thick of this material

may be substituted for the sand within the profile of the bank and for a width of 10 feet below the bottom of the berm, and a layer of stone ballast should then suffice for soling. Alluvial clay or other material, which after having been dried is found to be soluble in water, should not be used. Failing an insoluble clay, it is suggested that for class A rivers, a soling laid directly on the sand of 7 inches of broken-stone ballast should be sufficient, increased to 8 inches for class B rivers and to 9 inches for class C rivers. The soling should be covered, for all classes of river, by a uniform thickness of 3 feet 6 inches of pitching stone hand-set to reduce voids, so far as that can be done without any dressing of the stones. Quarry-refuse has been found to be unreliable in quality and quantity, and is not recommended for important works.

Berm.

As any movement in the slope-protection will destroy its efficiency, the design must embody some feature to ensure its permanency. This it is proposed to provide in the form of a level berm at the foot of what thus becomes the permanent slope. The width of the berm proposed is 15 feet for class A rivers, 20 feet for class B rivers, and 25 feet for class C rivers. The berm will consist of an extension of the apron. Before the pitching stone is placed in the berm, the ballast bed and the 3-foot 6-inch thickness of hand-set pitching stone should be extended from the foot of the slope for a distance of 10 feet over the bottom of the berm excavation. In order to ensure the permanent slope against guttering from the monsoon rainfall, a banquette of earth should be carried along the top of the permanent slope so that rain falling on the top of the bank would run off by way of the back slope.

Prospective Slope.

For the prospective slope the problem is to determine what thickness of pitching stone will provide at least an equal resistance to the washing-out of the sand through the voids as is provided by the soling and pitching stone detailed above for the permanent slope. All that is known on this subject is that at the Hardinge bridge a 4-foot 6-inch thickness of apron adjacent to the permanent slope proved to be insufficient at the head of the right guide-bank, and that where the apron had dropped into place at a 1-in-2 slope the resulting 3 feet of covering was still less adequate. The thicknesses shown in Table II (p. 175) for the prospective slope have been arrived at empirically by taking the 3-foot 6-inch thickness of pitching stone of the permanent slope as a basis, and adding 33 per cent. for absence of soling in all three classes, 11 per cent. and 22 per cent. to class B and class C respectively for magnitude of discharge, 11 per cent. and 22 per cent. to class B and class C respectively for high silt-content of the sand, and 22 per cent. to compensate for the exposed position of the head of the guide-

bank in all three classes. The idea will be followed easily in Table II, and it may be said that no accuracy is claimed for the percentages, which are merely used to draw attention to some of the conditions which appear to govern the problem. The result of the additions is that the basic thickness of 3 feet 6 inches is increased for the body and tail of the class A guide-bank by 33 per cent. to 4 feet 8 inches, and for the head of the class C guide-bank by 100 per cent. to 7 feet.

Apron.

The thickness of the covering of pitching stone for each case of prospective slope having been determined, it only remains to put the quantity of stone on the slope into the form of an apron of width 1·5 D and uniform thickness 1·5 T_1, which will be found in Table II.

Effect of Modifications on Quantity of Pitching Stone required.

The effect of these modifications in the design of the apron on the quantity of stone pitching required and on the security of the structure for the three classes of river will be seen from Table III, where the quantity

TABLE III.—COMPARISON OF QUANTITY OF PITCHING STONE AND MINIMUM THICKNESS OF APRON FOR BODY OF GUIDE-BANK AT SAME DEPTH OF WATER.

Class of river.	Tapered apron:	Apron of uniform thickness:	
	Quantity of stone and minimum thickness.	Quantity of stone.	Minimum thickness.
A	100	106	200
B	100	121	229
C	100	141	264

of stone as well as the minimum thickness in the section of the tapere[d] apron is represented by 100. The comparison is made on the dimension[s] for the body of a guide-bank for the same depth of water below apron i[n] each case. The depths taken were 40 feet for class A, 70 feet for class B and 100 feet for class C rivers. It will be seen that, although, in order t[o] double the thickness of the apron at its junction with the permanent slope the quantity of stone required for class A rivers only exceeds tha[t] required for the tapered apron by 6 per cent., the excesses in class B and class C are 21 and 41 per cent. in order to attain the increases o[f] 129 and 164 per cent. in the minimum thicknesses of apron whic[h] have been shown to be necessary. All the rivers which have bee[n] bridged up to the present, with one exception, have been class A rivers and the bridges in northern India which remain to be built are those i[n] the larger classes which have not yet been attempted. These increase

in the amount of pitching stone required for the body of the guide-banks do not cover either the percentage additions to be made to the deepest known scour for the body and tail, the greater additions for the head of the guide-bank, or the additional thickness of stone required to combat the severity of the attack on the heads of the guide-banks of these large rivers.

Alternative Overall-Apron Design.

The normal-apron diagram (Figs. 10 (*b*), Plate 1) has been drawn out, for the same "deepest known scour" and to the same scale as the tapered-apron diagram, for the purpose of comparison. Both of these diagrams are based on dimensions for class A rivers. The section of the head of a guide-bank suitable for the Hardinge bridge has been drawn out to the same scale, from the normal-apron diagram and Table of thicknesses, for a "deepest known scour" of 100 feet plus 90 per cent. This section therefore provides a means of comparing sections of class A and class C river guide-banks, and the difference in magnitude of the slope below the apron is striking. The latter section (Figs. 10 (*c*), Plate 1) also shows how much more is expected of a class C river in converting the apron into the evenly distributed protective covering of the lower slope. In addition, it will be noticed how insignificant a part of the fully-developed section is occupied by the permanent slope.

Examination of the completed design confirms the view of the Author that the conception of a permanently pitched upper slope was a mistake. It is not possible in these rivers, and especially in the large rivers, to assess so accurately a maximum scour of rare occurrence as to ensure that the permanent slope will not be cut into by small slips travelling back from the toe of the developing apron in the natural course of events. So long as the slips take place within the apron, as soon as the exposed face of sand begins to be eroded sufficient stone drops into position to seal the exposed face, but where the slips occur in the permanent slope the protective cover is insufficient for this purpose, and if the slip remains unnoticed disaster follows. In the completed design it was sought to avert this danger by introducing an extension of the apron, under the name of the "berm." There is no certain permanence about the berm, but it does point the way to a solution of the difficulty, for seen in the diagram it is evident that the berm-apron would go a long way towards providing a flatter slope of 1 in 3 with an apron which could be extended to the top of the bank without any great increase in the quantity of stone required. In fact, it would probably entail less expenditure than the costly composite covering suggested in the completed design. In class C rivers, where the apron is likely to be laid at about half-flood level, there would seem to be no doubt about that point, and it would probably suit all rivers. A slope apron of thickness detailed in Table II would be sufficient to pro-

tect the bank if it were laid upon bare sand, and if in coming into actior as an apron the slope should be steepened to 1 in 2 it would seem that th thickness of cover would become some dimension between 1·5 T_1 and T_1 which, being greater than T_1, might be considered to be sufficient to provid the greater protection needed for water-surface disturbances in the uppe part of the slope. It is possible that for ordinary waves it is the hori zontal dimension of the covering of pitching stone which would influenc movement of the underlying sand, but it would seem that this horizonta thickness would have no restraining effect on the surge-waves with periodicity in the vicinity of 2 minutes. However, as any exposure o the sand face would be followed immediately by a fall of the apron an staunching of the exposed surface, and as severe examples of surge-wave would not be likely to last longer than 12 hours, it is not considered tha these need cause anxiety.

In the early stages of the bund-and-apron system the slope of the fac of the bund was 1 in 2, but reserve stone was placed on the slope-stone bringing the face-slope of the stone to 1 in 1. The quantity of stone i the early aprons being usually insufficient, as soon as the slips in the apro extended deeply into the bund the mass of reserve stone would drop int the deep scour and be lost. The present proposal to use the flat slope o 1 in 3 covered by an apron of the greater thickness now considered necessar is therefore not a return to a practice previously tried and discarded, bu is an extension of the well-tried automatic-apron principle to the slop between high- and low-water levels where the resultant additional thicknes of covering would take care of water-surface disturbances. An overa apron has much to recommend it. It is a great constructional advantag that it requires the use of only one material. It is a form of constructio which has no line of weakness such as occurs at the change from the aut matic slope-pitching apron to the permanent slope-covering. Finall in its extreme form as an apron laid on the natural ground at about high flood level it has been successfully employed by Bell to limit the encroach ment of a cutting bend in the tidal part of the Karnafuli river at the port c Chittagong.

In both Figs. 10 (*b*) and 10 (*c*), Plate 1, the overall apron is shown a an alternative construction. This form of protective covering is offered a an alternative to the permanent slope, berm and apron design, that of th apron proper being common to both. In general it would seem that th overall apron is better suited to cases where the apron would necessaril be laid at a high level, but that where a low-level apron is possible con parative designs should be prepared in order to assist a decision. It ma be observed that the overall-apron design offers security against each the three causes which have been put forward to account for the breac in the right guide-bank of the Hardinge bridge: namely, inadequacy soling for the pitching of the permanent slope, insufficiency in thickne of apron at the foot of the slope, and insufficiency in width of apron.

Relation of Guide-Bank Apron to Adjacent-Pier Apron, and Depth of Foundation of Adjacent Pier.

Although this is merely a particular case of the general principle of avoiding the formation of eddies in proximity to piers, it is sufficiently important to be re-stated as emphatically as possible in a different form, as follows. The guide-bank apron should never, in any circumstances be extended locally to enclose the adjacent pier. The inadvisability of a stone connexion between the abutment and the next pier was first pointed out by Bell about 60 years ago at the Empress bridge [1] before the bund-and-apron method had been formulated. The effect of enclosing the adjacent pier in the apron is to prevent the part of the apron between the pier and the abutment from dropping into position, with the result that an inerodible ridge remains, forming with the stone around the pier a submerged spur. As soon as there is any considerable flow of water along the guide-bank and the toe of the apron which has been carried round the pier begins to drop, all the conditions for the formation of an eddy of maximum size are fulfilled. The eddy develops below the bridge between the pier and the guide-bank, eating into the guide-bank and threatening the safety of the approach. It scours at high flood and it will continue to scour, in any river with a bed of fine silty sand, at low water when the surface-movement is hardly perceptible. The eddy in this position is a perpetual menace. The ridge is too near the bridge to be removed by explosives and, where the crest of the ridge is well below low-water level, its removal by dragline excavator is problematical. The cost of maintenance of the works in the vicinity of the eddy is prodigious. Much can be done in designing the bridge to make it difficult for the connexion to be made by ignorance or inadvertence, and the following rules to this end are suggested.

The full-depth abutment-pier should be planted in the apron at the foot of the slope. This will ensure the greatest distance possible between the outer edge of the apron and the adjacent pier. The adjacent pier should have its own separate apron, as shown in *Fig. 11* (p. 184). The adjacent pier should have a deeper foundation than is considered necessary for other piers in the bed of the river to allow for the deeper scour that is to be expected in the vicinity of the guide-bank. The intention of these rules is to ensure smooth flow along the guide-bank.

It may be mentioned that the precaution taken of showing the guide-bank apron as quite independent of the adjacent-pier apron in the original design (see Fig. 10, Plate 2, of the Author's previous Paper on the Hardinge Bridge [2]) has proved to be ineffective.

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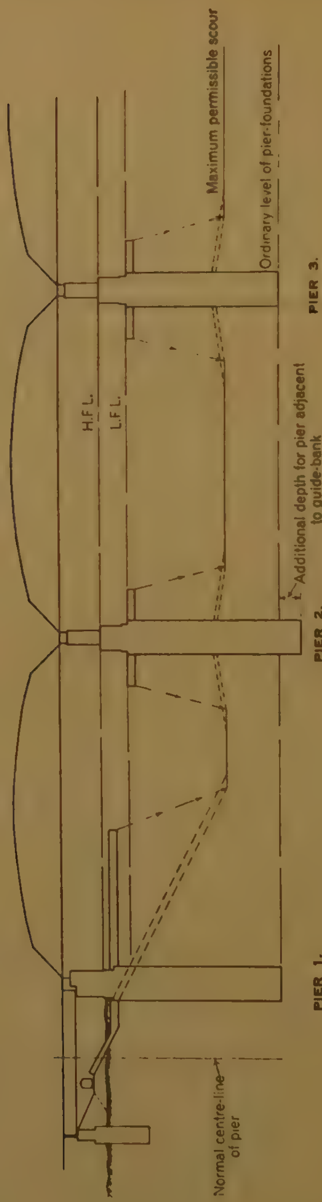
[1] W. Macrae, "Training in Connection with the Shortening of the Empress Bridge over the River Sutlej." Minutes of Proceedings Inst. C.E., vol. 237 (1933–34, Part 1), p. 119.

[2] "The Hardinge Bridge over the Lower Ganges at Sara." Minutes of Proceedings Inst. C.E., vol. ccv. (1917–18, Part 1), p. 18.

Fig. 11.



POSITION OF PIER 1 TO ENSURE FULL DEVELOPMENT OF GUIDE-BANK APRON INDEPENDENTLY OF APRON OF PIER 2.

General Design of Bridge.

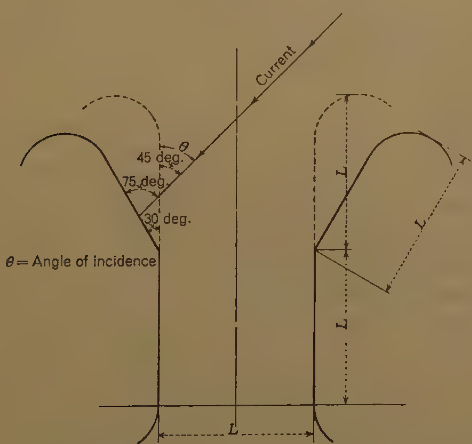
Guide-bank bridges have hitherto consisted of a series of spans of equal length, but consideration of the problem stated on p. 183 leads to the conclusion that the end spans should be sufficiently longer than the intermediate spans to remove all danger of any connexion of the guide-bank apron with the adjacent-pier apron. The adjacent pier should nevertheless be given a greater depth of foundation, in view of its special position with regard to the guide-banks, than has been proposed for the rest of the bridge.

The length of end span required in class C rivers in order to give the apron some possibility of independent normal development, and to remove the adjacent pier from the region of deep scour along the toe of the guide-bank apron, will entail a cantilever bridge, which, having fewer piers, will require deeper foundations throughout.

Disadvantages of Splayed Guide-Banks.

It has been stated (p. 165) that guide-banks of upstream length equal to the waterway should in no circumstances be splayed, on account of the increased obliquity of current at piers to which this may give rise. From

Fig. 12.



One Disadvantage of Splayed Guide-Banks.

this it might be inferred that there was no objection to splaying extensions above that length. This, however, is by no means the case, as may be seen by referring to *Fig. 12*, which shows the disadvantage of the splay in diagrammatic form.

In this diagram the angle between the direction of the current and the

line of the bank is termed the "angle of incidence." It has been found that depth of scour and damage to bank is least when the angle of incidence is zero and the current is parallel to the bank, and that the scour and damage are greatest when the angle of incidence is 90 degrees—that is when the direction of the current is normal to the bank—and it is assumed that scour and damage increase with the increase in the angle of incidence between these limits. It will be seen that if in the diagram the angle of splay is 30 degrees and the angle of incidence of the current on one of the parallel guide-banks is 45 degrees, the angle of incidence on the splay is 75 degrees. It follows that a splayed guide-bank needs to be more heavily protected than one that is not splayed.

Part III.—THE TRAINING WORKS OF THE HARDINGE BRIDGE

Introduction.

In his Paper on the Hardinge bridge over the Lower Ganges at Sara, the conditions of the problem of bridging the river, as they then appeared to the Author, are described under the headings of site, training works and main features of the bridge.

Site of Bridge and Original Lay-out of Training Works.

The site selected for the bridge has been indicated in *Fig. 13*, which shows the limits of wandering of the Ganges between Sarda and Pabna during a period of 128 years between 1781, the date of the first map, and 1909 when the site of the bridge was determined. During the whole of this time the river had been controlled and held by the Sara clay formation on the left bank and the Raita clay on the right bank. The head of the Raita peninsula, by means of cut-offs, had been eroded from the dotted line of the river in 1780 to the line of its present bank, and the Sara clay had similarly been evenly eroded to the extent, variously estimated, of perhaps ½ mile. Although the Ganges is stated to have broken across the line of flow of the rivers running southward from the Himalayas in the sixteenth century, it is discernible from Rennell's map of the delta published in 1781 (*Fig. 1* of the Paper previously alluded to [1]) that it had not attained to its present magnitude below Raita in the period of some 200 years from the sixteenth century to 1781. This is evidenced by the long sweeping

[1] Footnote (2), p. 183.

curves in the course of the Ganges above the off-take of the Jalangi river and the greatly increased amount of curvature in the river below that point. A very large part of the Ganges discharge must still have been finding its way *via* the Jalangi and the Bhagirathi to the Bay of Bengal by way of the Hooghly. The great amount of curvature below Raita is

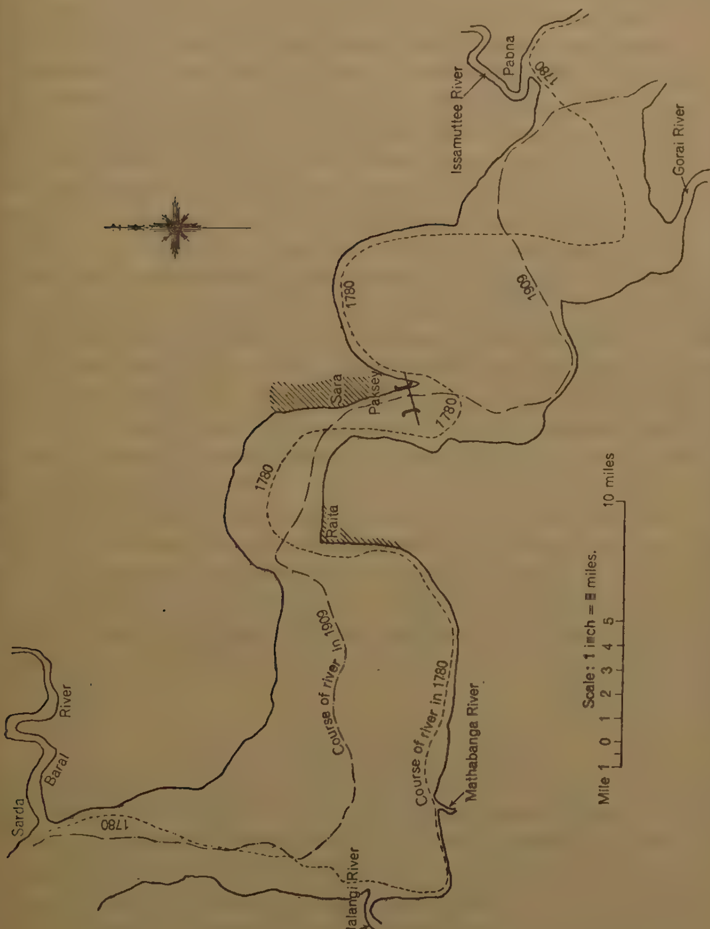


Fig. 13.

Limits of Wandering of the Lower Ganges in the Vicinity of Sara between 1780 and 1909.

very clearly shown in *Fig. 13* (above) by the double-S curvature in the immediate vicinity of Sara where now a single S-curve suffices. It would seem that, during the time of the double curvature, the Ganges there had been silting up its banks and cutting through the bottom of a depression, extending south-westward from the Chalan Beel, which had not previously been built up by the Ganges or the Brahmaputra. It was no

doubt the sedimentary deposit in this old depression, indurated, as it was found to be, in a belt just above low-water level at Sara and at Raita, which had gained such a reputation for stability under the name of the Sara clay.

In the search for a site in the vicinity it was found that the river had remained sufficiently constant at Sara to allow the ferry to be worked from that spot on the left bank from the earliest days of the railway; and it became apparent that a site below Sara, in view of the fair alignment and reputed stability of the left bank, offered many advantages. In particular, it appeared to offer immunity from any embayment of the river at the back of a flanking guide-bank on either the right or the left bank of the river. In other words, the river appeared to be held in complete control by the Sara clay, subject only to a periodical sequence of bend and cut-off between Sara and Raita. In these favourable circumstances it was considered that there would be no necessity for the upstream length of the flanking guide-banks to exceed three-fifths of the waterway, and no need for the heads to be curved back more than 60 degrees. The exposed alignment of the railway on the left bank of the river appeared to be completely protected by the resistant Sara clay, and the site about 3 miles below Sara was finally selected. A site had thus been found at which there appeared to be no prospect of the condition arising of the river issuing from a deep embayment and crossing the head of one of the flanking guide-banks, to strike the other in the vicinity of the bridge. This was considered to be the most trying test to which the bridge and training works of a class C river could be subjected, and was sometimes referred to as "the triple attack." The training works provided at the site 3 miles below Sara consequently consisted of a pair of flanking guide-banks at the bridge-site, each 4,000 feet in length; a revetment of the bank at Raita, 4,000 feet in length; and a similar revetment at Sara, 3,650 feet in length.

History of Training Works.

The flanking guide-banks, named the "Right" and "Left" guide-banks, were completed, for all practical purposes, in 1911, and the behaviour of the river had been sufficiently satisfactory up to 1924 to allow a large reduction in the quantity of pitching stone kept as a reserve for repairs. Any complacency, however, which might have been felt at this satisfactory state of affairs was rudely shattered by the receipt of a letter dated the 23rd June, 1932, from India asking the opinion of the consulting engineers on a report and estimate for an addition to the training works at the Hardinge bridge. The report disclosed that in 1925, the year after the reduction in the reserve of pitching stone, the river had moved over into the Lalpur bight and, having developed a very severe attack on the upstream end of the Sara protection-bank, had nearly got behind it. The

attack had continued year by year, and by 1932 the river had eroded the clay bank above the end of the protection-bank to a depth of embayment of ¼ mile and was running across the partially submerged wreckage of the end of the protection-bank in a direction which threatened a deep embayment above the head of the right guide-bank. The spur-condition which had thus arisen at the upstream end of the Sara protection-bank was accompanied by the usual eddies, one upstream and one downstream of the spur, of which the latter had reached extraordinary dimensions, and the eddies were driving the main stream abruptly across to the right bank of the river where unexplained damage was taking place at the head of the right guide-bank and unexpected depths of scour were being experienced downstream of one of the piers of the bridge. In these circumstances the consulting engineers recommended the construction of a guide-bank at Damukdia on the curved dotted line connecting Raita protection-bank with the right guide-bank (shown in Fig. 14, Plate 2), and suggested that an attempt should be made to restore smooth flow at Sara by cutting down the projecting remains of the sunken head and curving back the protection-bank to a tangent, making an angle of 60 degrees with the axis of the bridge. These works were successfully carried out, and Fig. 14, Plate 2, shows the state of the training works in 1933. The consulting engineers were also able to direct attention to the existence of a band of clay, passed through in sinking piers 4 and 5, which appeared to supply an explanation of the scour downstream of the pier. The main stream of the river, driven across to the right bank by the whirlpool at Sara, was attempting to excavate a channel through a patch of clay which had escaped or successfully resisted erosion when the river had passed over it on previous occasions. The clay patch, explored by borings, was found to be of great extent. At its highest level between piers 4 and 5 it was 42 feet below low-water level and 15 feet thick, consisting of 4 feet of blue and 11 feet of black clay. The material below the clay was described as coarse sand with gravel down to 159 feet below low-water level. In the borings near pier 2 fine black sand and coarse blackish sand are also mentioned. These sands, which must have been brought down direct from the Himalayas by some old river, bore no resemblance to the fine white sand of the Ganges. The clay became thinner towards the limits of the patch and consisted of 3 feet of blue clay only, at a depth of 77 feet below low-water level, opposite chainage 4 of the right guide-bank. The clay was proved by borings from pier 1 to pier 6 and for 900 feet upstream, but there was very little of it downstream of the bridge. The clay patch, with its sub-stratum of sand, has been described in some detail on account of its unfortunate effect on the course of events at the guide-bank. In the floods of 1933 the main stream, diverted from further embayment by the lower part of the newly-constructed guide-bank at Damukdia, came down on the head of the right guide-bank, and, passing along the face of the head, was divided by the ridge of the clay patch into a

stream running obliquely towards No. 6 pier and into what appears to hav been the main stream running along the toe of the guide-bank through spa No. 2; nevertheless, no great depths of smooth-flow scour had bee recorded in this channel. In the years before 1933 slips had occurre in the upper slopes of the head of the guide-bank due to insufficien thickness of pitching stone, but although some scour had occurred a the toe of the apron-stone, the stone had not followed down. In 1933 as described and discussed elsewhere, the breach occurred and the balanc of evidence appears to be that if scour took place the apron did not follow down before the breach. It was not until after the floods of 1934 that th catastrophic slip took place between chainages 5·5 and 10·5 adjoining th breach, and here again the apron-stone had not followed the scour down Continuing along the guide-bank, it will be found that the adjoining lengt of apron between chainages 1 and 5·5 has not yet gone down at the tim of writing, and sections and borings taken after the floods of 1936 showe that the apron was held up by the clay-patch strata including the 4-foo bed of blue clay, and that scour in the channel had not reached a greate depth than 109 feet below low-water level. The behaviour of each par of the guide-bank above the bridge is so similiar that it is reasonable t suppose that all of it is underlain, as has since been verified, by clay-patc strata, and it is possible that in compensation for the delay and uncertaint in the development of the apron, there may be found some limitation due t these strata, in the depth of smooth-flow scour near the bridge in this channe

After the immediate safety of the bridge had been secured by th works carried out at the breach, in 1934, the Railway Board, alarmed b these unexpected and, at that time, unexplained occurrences, and at th cost of the repairs, appointed a committee of engineers to examine th training works and to prepare a comprehensive scheme to ensure th proper working of the existing training and protection-works.

The Committee's Proposals.

In 1935, rather over a year later, the Committee, assisted by mode experiments carried out at Poona, submitted their final recommendatio to the effect that "the protuberance at Sara should be removed," tha "the guide-banks at the bridge should be lengthened to approximatel the length of the bridge," that "the Damukdia guide-bank should b removed" and that should the Lalpur bight embayment threaten the mai line this should be retired if necessary by from 1½ to 2 miles from Abdulpu to the bridge.

These somewhat sweeping recommendations appeared to confli with the terms of reference, but shortly after their submission orders we issued to strip the protection-bank of pitching stone and to proceed wit the removal of the Sara protuberance. The Committee's proposals ar indicated in Fig. 15, Plate 2.

Accepting entire and sole responsibility for the design of the training works up to the point reached in Fig. 14, Plate 2, and finding the ideas embodied in the Committee's recommendations so completely opposed to his own, the Author studied with care the two volumes (Part I: the Report, and Part II: the Model Experiments) and came to the conclusion that the point of divergence lay in the application by the Committee of the opprobrious name of "protuberance" to the Sara protection-bank, or rather to the Sara key position. The explanation of this misnomer has only lately become apparent. The Committee at their first meeting laid down on the map a red line, in continuation upstream of the left guide-bank, which curved back in an easy sweep corresponding to the curved dotted line on the right bank shown in Fig. 14, Plate 2. This curved line passed at the back of Sara and Sara became a protuberance so far as that line was concerned, and was blamed for all the unexpected happenings at the right guide-bank. Although it is now known that these untoward events were due to faults in the guide-bank (particularly to the unstable character of the sand core, the insufficient thickness of the protective covering of stone pitching, the presence of "clay patch" strata extending under the guide-bank, and the eddy due to the connexion of the guide-bank apron with the apron of pier 2), and although no use was made of the curved red line, which might well have been erased, the idea persisted which has resulted in the recommendations to dismantle and abandon the Sara key position and the Damukdia guide-bank. Fig. 16, Plate 2, showing the relation of the Sara key position to the line of the axis of the river, displays the Sara position in a different light as the head of a guide-bank.

The Author's Proposals.

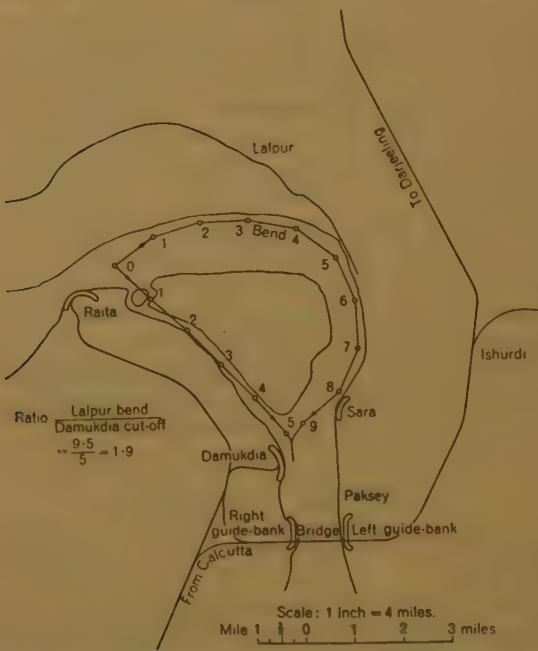
The proposals submitted by the Author appear in diagrammatic form in Fig. 17, Plate 2.

In a survey of the river dated the 20th October, 1936 (shown in *Fig. 18*, p. 192), there appeared the first definite indication that a cut-off of the Lalpur bend by way of the Damukdia channel might be expected. This was an important event because it showed that, as long as the Sara key position remained in existence, any embayment arising from the river passing Raita towards the main line between Gopalpur and Ishurdi and returning past Sara would cut-off *via* the Damukdia channel before the main line was threatened. The bend and incipient cut-off are shown on *Fig. 18* where a ratio of bend to cut-off of 1·9 is indicated. This is slightly larger than the ratio of 1·75 at which the Golbathan cut-off was established, due no doubt to the chord-channel leaving the main stream at right angles in this case.

The extent to which the maintenance of the Sara key position may be said to provide a final solution depends upon the following considerations. The Damukdia cut-off, indications of which had first appeared in 1934

before the pitching stone had been removed from the Sara protection-bank, was reported to be taking 27 per cent. of the discharge with a flood-level at the bridge of 245·6 in the early part of September 1937. With even moderately high floods the opening out of the cut-off channel should now proceed rapidly, and no further great amount of erosion is likely to take place in the bend of the river, which at Dhapari is at present no less than 2 miles from the main line of the Railway. The old bank of the Lalpur bight (Fig. 17, Plate 2) has been eroded from Sara to a short distance

Fig. 18.



LALPUR BEND WITH DAMUKDIA CUT-OFF: SARA KEY POSITION IN ACTION. (COURSE OF RIVER IN OCTOBER, 1936.)

beyond Dhapari, but from this point onwards to Lalpur and Bilmari it has not been attacked. The old bank corresponds to the position reached by the river in 1868 and 1893, and it has been raised, naturall by flooding and artificially by the ring of villages built upon it, to a lev which has long prevented any serious spill of Ganges water across i Between Sara and Dhapari the average level of the river-bank is abov high-flood level, but the ground has a cross fall towards the railway of 2 c 3 feet per mile, as is usual with alluvial rivers which flood their bank Between Dhapari and Lalpur and onwards the levels are missing in th Committee's Report, but it is probable that they are similar to tho

between Sara and Dhapari for at least a mile from the bank, but that beyond that distance there is a steeper fall towards the low ground between Gopalpur and Abdulpur, where at the Bowlah Khal there is a spill opening in the railway line. Where the old bank has been eroded, as in the vicinity of Dhapari, as soon as erosion ceases the villages will no doubt be rebuilt and the bank of the bight will return to its former condition. The main river will have occupied the Damukdia channel 3 miles away, and, subject to the fulfilment and verification of the aforesaid expectations, a period of freedom from anxiety on account of Ganges spill, from the Lalpur bight into the Chalan Beel, will follow. The Author has hitherto assumed the cycle of bend and cut-off between Raita and Sara to occupy about 33 years, and a close examination of the more recent records shows that this would be better expressed by saying that the length of the cycle may vary from 30 to 40 years. Whatever the length of the interval may be, if the river on its next incursion into the bight passes Raita on the same course as it does to-day, it may confidently be expected to cut-off before any further erosion of the high land in the neighbourhood of Dhapari would take place; and if the river passes Raita in a northerly direction as depicted in the plan of the river in 1868, it may equally confidently be trusted to cut-off before any serious erosion could be effected in the northern part of the old bank of the bight. These predictions are based on the knowledge that similar cut-offs have taken place before, and they are subject to the proviso that no changes shall have occurred in the conditions which govern such cut-offs. The principal governing condition for the repetition is that there shall have been no change in the distance between the west face of the Raita protection-bank and the face of the Sara protection-bank. This distance limits the length of the cut-off chord, which in turn limits the length of the bend and consequently limits the depth of penetration of the old bank. A proposal has lately been put forward to construct a left guide-bank, splayed at the back of Sara in such a manner as to amount to a retirement of about $\frac{1}{2}$ mile from the original Sara revetment. This would seem to allow of the bend making an additional penetration of $\frac{1}{2}$ mile in the bank above Sara and from $\frac{1}{2}$ mile to 1 mile in the northern part of the old bank. Such a penetration as the latter would probably enter the low ground and would necessitate the construction between Ishurdi and Abdulpur of a retired dam with sluices, to control the spill into the Chalan Beel.

In the preceding paragraphs the importance has been established of maintaining the position at Sara as closely as possible, and with regard to that it may be said that, owing to the superior resistance of the clay on the river face, the damage done is not yet irretrievable, and it appears that the position could still be restored with but slight retirement of the guide-bank head, which would be connected with the original left guide-bank by means of an "interrupted" guide-bank. There would, however, be little difficulty in connecting the slightly retired head directly with the

original left guide-bank, the connexion passing across a well-silted-up embayment of no great depth.

In addition, a probability has been indicated that after a moderate attack on the partially extended left guide-bank, the main stream from the Damukdia channel will flow quietly down through the bridge for the next 30 years, and, but for the damage done at Sara, with comparatively little further expenditure on the training works.

The only place where trouble appears to be inevitable is at the eddy downstream of the bridge between pier 15 and the left abutment, caused by a connexion between the guide-bank apron and the pier-apron. The connexion so far as known consists of a single layer of pitching stone and, if deep scour is permitted at the toe of the guide-bank apron upstream of the pier and at the pier, the obstruction would probably clear itself by slips upstream of the pier. The resulting deep flow between the pier and the abutment would then extinguish the eddy. It would be prudent to place a reserve of pitching stone on the lower part of the slope of the guide-bank and the inner belt of the apron in case slips should occur in the bank in the course of development.

It is even more necessary that steps should be taken to remove the more heavily pitched apron-connexion between the right abutment and pier 2 as, until this is done, there can be no security when the main stream returns again to the right bank. It is probable that this could be done by removing the pitching stone upstream of the bridge from the apron which has not yet gone down, and cutting down the sand to low-water level. The pitching stone on the connexion with the pier-apron, being thus loosened up at the edge, could then be removed by grabbing with powerful plant operated on shore or afloat.

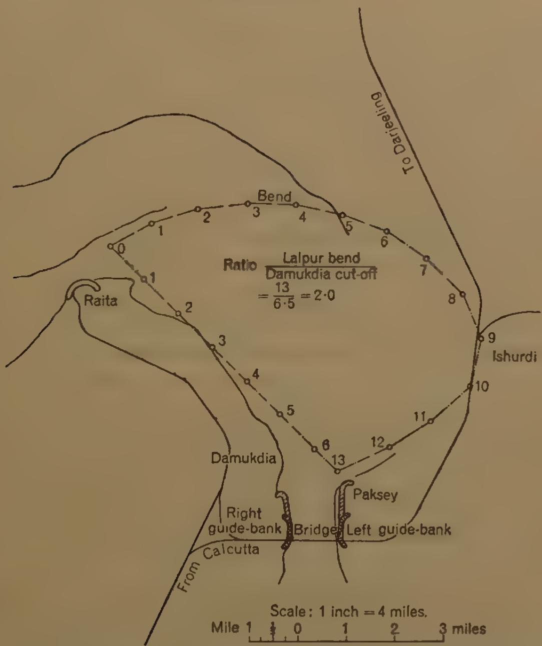
Examination of the Committee's Proposals.

In *Fig. 19* is portrayed the probable development of the Lalpur bend assuming that the Sara key position had been eroded. Cutting-off at a ratio of 2·0, it would have necessitated the retirement of the whole of the main line from the bridge to Abdulpur. It will also be seen that the main river, crossing the head of the left guide-bank proposed by the Committee, would attack the right guide-bank immediately above the bridge. This is the form of attack which is most to be feared in the case of a river of the magnitude of the Lower Ganges, and it was to avoid the possibility of such an attack that the bridge was sited below Sara and that Sara was protected and made permanent. There is a particular reason to fear the attack at the angle between the bridge and the right guide-bank, in the whirlpool between the guide-bank and pier 2 caused by the connexion between the guide-bank apron and the pier-apron. There is an objection to the retirement of the main line towards the low land of the Chalan Beel which in flood-time extends across the Sara-Serajganj railway and affords

direct water-communication with the Brahmaputra. Between Sarda and Pubna, Sara is the only stable feature on the left bank and it has stabilized the river there for more than 150 years. It is impossible to predict what the effect of its removal on the regime of the river would be, but the forces released might well be the cause of uncontrollable changes.

The conclusion was reached above that on the full development of the Damukdia cut-off, brought about by the Sara protection-bank, the river might be expected to flow quietly down through the bridge without giving

Fig. 19.



Probable Development of the Lalpur Bend, assuming the Loss of the Sara Key Position.

ny cause for anxiety for the next 30 years; and if this is considered in elation to the Committee's proposals, the difficulties and delays attending he execution of any scheme, which is dependent on the successive demoliion of existing works, become only too apparent. The extension of the eft guide-bank, which is common to both schemes, is actually in hand, ut the extension of the right guide-bank must await the silting up above ow-water level of the gap between it and the Damukdia guide-bank. It as not considered safe to remove the latter before lengthening the right uide-bank, and it is uncertain by what date this will become possible. his, however, is unimportant, provided it is completed before the river

next embays above Sara. The head of the lengthened right guide-bank in a more exposed position, must then be prepared to meet an attack coming from the wreckage of the greater part of the Sara protection-bank of equal intensity with the earlier attack due to the wreckage of the up-stream end of the protection-bank, on which the Sara position was condemned.

Addendum to the Author's Proposals.

Since the Paper has been prepared, report has been received of a Ganges spill from the Lalpur bight in the latter part of August, of sufficient magnitude to determine the Author to call attention to the two proviso on p. 193, with a view to a modification of his proposals in one respect.

The report states that "The left bank between Bangalpara and Dhapar was completely submerged during the high flood. The spill water passing through the Lalpur channel inundated the country before flowing through the main line culverts and the water level on the upstream side of the main line between Gopalpur and Abdulpur reached the formation level." No such flooding has come to the knowledge of the Author during his long acquaintance with Sara, and it is possible that it has never occurred before. When this possibility was previously considered (see Appendix IV paragraph (b) of the Author's Report on the Hardinge Bridge, dated December, 1933–January, 1934), the Author recommended the construction of a levee or marginal embankment to start from the old Sara metre-gauge bank and follow the curve of the 1868 bank of the river, at about 1 mile distant from it, to abreast of Bilmaria, and it would seem that if this had been carried out the flooding of the main line to formation-level would have been avoided.

The submergence of the left bank appears to have been brought about partly by the direction of flow and concentration of the river at Raita, and partly by the destruction of the upstream portion of the Sara protection bank. Confirmation of this conclusion is supplied by the change in the ratio of bend to chord from 1·9 when indications first became definite to 1·6 at the present time. Nevertheless, to show by what a narrow margin safety was missed, the Report may be quoted as follows: ". . . if the main stream at the Raita protection bank straightens a little more and presses against the protection bank about 400 feet further downstream before the flood subsides, in all probability the Damukdia channel will open this year."

The serious results which have followed from a lateral erosion of 800 feet at the curved part of the dismantled Sara protection-bank, and the fact that any practicable restoration would entail further retirement, are sufficient to show that the position has been lost, from the point of view

of limitation of bend-erosion by cut-off, and that the altered circumstances demand a review of the situation.

Briefly, it becomes necessary in the special conditions which have now arisen to withdraw from dependence on the bend and cut-off principle alone, and to revert to the provision of a retired line furnished with sluices crossing the spill between Ishurdi and Abdulpur, as proposed by the Committee, and more fully described in the Author's Note on the Report of the Hardinge Bridge Committee, dated July, 1936. This combined railway line and dam is an adjunct of the Author's scheme of training works which is no way modified except so far as may be necessary to facilitate the construction of the extension of the left guide-bank on an alignment inclined so as to pass some 300 feet to the back of the downstream leg of the Sara protection-bank, thereafter continuing on a curve of 1,910 feet radius to form a Curzon head, as previously projected.

The advantages of the scheme, which remain as before, are :—

(1) The removal to as great a distance as possible from the bridge of the great depths of scour found at the heads of these guide-banks, which at Raita and Sara have approximated to 200 feet below low-water level.

(2) The protection of the main line from the bridge to Ishurdi.

(3) The restriction of curvature so that the river will remain pinned at Sara as it has been for the last 150 years, and will not be free, as with a shorter guide-bank, to set up new oscillations extending both up- and downstream which would adversely affect not only the bridge but other important interests.

(4) The scheme ensures, after the closure of the gap between the right and Damukdia guide-banks, that the river will always have a straight run through the bridge for which alone it was designed. The expectation is that when the river, in the next cycle, begins to embay at the back of the new Sara head, and the river changes across from the left to the right bank, the main stream will move over in a bend impinging first on the upstream part of the "closer," whence it will widen out and pass straight down through the right half of the bridge. This will take place without the concentration at any time of the whole river upon the angle between the guide-bank and the bridge, as would be the case with a left guide-bank of length equal to the length of the bridge or with a longer splayed guide-bank. The splayed form of the consolidated right guide-bank requires that the "closer" should be constructed to the design for the section of the guide-bank head delineated in Figs. 10 (*c*), Plate 1. At each end of the "closer" it is essential that provision should be made for the free passage of water, for the double purpose of silting up the low ground at the back and of balancing the water-pressure on the front and back of the bank.

The adaptation of guide-bank principles to the safeguarding of the

Hardinge bridge and its approaches has necessitated enquiry into eve detail of the guide-bank system, and the Author hopes that the Gener Principles put forward in Part II of this Paper will be found useful by tho interested in river-training for railway bridges.

The Paper is accompanied by nineteen sheets of drawings, from whic Plates 1 and 2 and the Figures in the text above have been prepared, a by the following Appendix.

APPENDIX.

"The Continuous Bund and Apron Method of Protecting the Flanks Bridges for Rivers in the Panjab (India)." *

By James R. Bell, M. Inst. C.E., Engineer-in-Chief, Frontier Railway Survey, Ind

"In the Panjab Rivers—which erode their banks and scour their beds deep on the outer edges of the erosive bends—our practice is to retain the stream within limited length of the Bridge by protective bunds faced and aproned with rough sto pitching. In cases where there is no solid ground on which the heads of these fla bunds can rest, we make their length up-stream at least equal to the length of t Bridge itself, and extend both of them down-stream beyond the abutments to a distan of at least one-fourth the Bridge length.

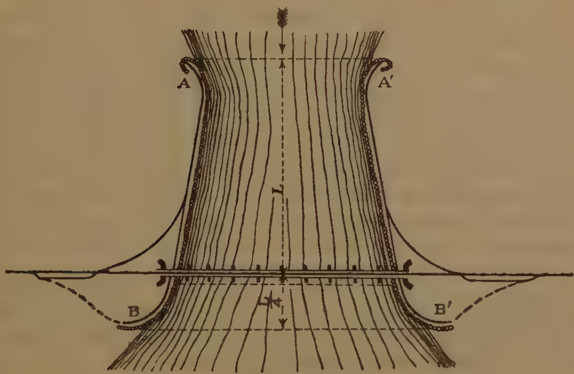
"2. The alignment that is thought best on abstract considerations is sketched Fig. I. In the absence of natural heads those at AA′, are strengthened by very lar mounds of stone, in cases as much as 300,000 cubic feet per head. The extent to whi the river should be throated between A and A′ depends on the number of piers in t bridge, as these so obstruct and subdivide the channel that a very much narrow width at AA′ gives a much larger effective channel than that afforded by the brid The object of the vena contracta on plan is to centre the river and make it fan o equally in all the spans. As a rule the conditions of the site do not admit of usi the vena contracta ground plan, and it is found that where the bunds diverge on t up-stream side there is a proportionate tendency for an island to form in the midd of the bridge which splits the deep channel towards the abutments. The dow stream tails of the bunds at BB′ are necessary to conteract the eddy that tends undermine the ground below either abutment.

"3. The cross section of Bund now in vogue is shown in Fig. II and the main fact in determining its proportions is the normal deep scour of an erosive bend referred as 'S' in the following portion of this note. This factor is not always easy to ascerta

* Reprinted from Technical Paper No. 2B. Simla, 1890.

nd should be carefully discriminated from the enormous depths attained in purely lluvial strata by eddies. For example, in the Chenab and Sutlej, eddy-scours of 60 nd even 70 feet below High Flood Level are known to occur, while 40 feet is the normal rosive scour (the S. of the diagram). Where rock is found overlaid by other than firm

Fig. I.

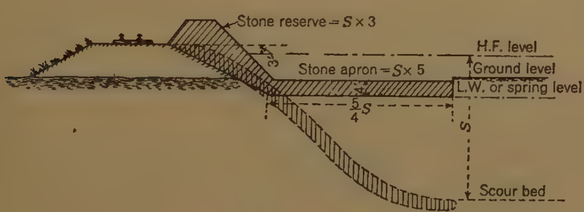


Normal Alignment of Bunds.

trata of probably considerable age, the rock is sure, sooner or later, to be scoured lean. At Sukkur the Indus cleans its rocky bed almost every year at 120 feet below ligh Flood Level. The dotted section shows the ultimate position of the apron when cour has engulfed it.

"4. After deciding on the centre line of the bund (which should in all cases be at east 20 feet wide on top with slopes not steeper than 2 to 1 from 3 feet above High

Fig. II.



Normal Section of Bund.

lood Level down to the spring level at which water is encountered when the river is w) the apron pit is laid out with a width from toe of slope at spring level outwards $= \frac{5}{4}S$. The bund is made wholly from the apron pit and if more earth is wanted it is ug from the river side, as borrow pits in rear of the bund are very objectionable, and able to induce ' blows '. Where the apron pit yields more earth than is absolutely quisite the width of bund is increased till they balance. The core of the bund should, possible, be of fine sand and the slopes of good clay; while the rear slope should e wattled and planted with willows, elephant grass, or other deep rooted vegetation, s protection against the lap of wavelets that arise on the lake, which forms in rear

of the bund by spill or percolation. In the Panjab this lake is purposely filled by controllable sluice inlet that brings in silt to warp up the lake bed. This process only effective when a high level outfall draws off the clean upper water and keeps up steady influx of silt during flood time.

"5. We now invariably lay the apron 4 feet thick, and hence its cross section area = $S \times 5$. The total amount of stone laid in at first = $S \times 8$ and the surplus reserve = $S \times 3$ is stacked on the river slope of the bund at as steep an angle as it w stand, usually a little steeper than 1 to 1. The top of the bund carries a tramway which the reserve can be transferred to any point where heavy scour threatens to eng the reserve stone already in place. During the first three or four years of its existen we think it essential to renew the entire reserve when the river is low, and even increase it when the indications point to our having underestimated the factor S.

"6. Sharp-edged stone is best; and round or even cubical pieces are found wastef and inefficient. The individual pieces should be of approximately one size and the weight ought to be the greater as the velocity of the stream increases. For 6 feet p second average velocity, stones averaging 1 cwt. suffice, if sharp edged and of hig specific gravity, to revet a subaqueous slope as far down as the scour extends. W attribute the success in point of stability and economy that has so far invariab attended this method as compared with that of stone faced spurs to the fact that th latter provoke and intensify eddy scours while the former tends to eliminate eddi and to straighten out and so minimise the attacks of bend scours.

"J. R. B."

25th August 1890.

Discussion.

*** **The Author** wished to draw attention to one of the first principles of river-training for railway bridges which, although always taken for granted by railway engineers, did not appear to be known to engineers who might only have had experience of river-training for other purposes. The general principle was that the pair of flanking guide-banks should converge upstream. It was not always possible to ensure convergence on both sides, especially if the river were on the move at the time of construction, but the knowledge of the principle would have prevented a serious mistake in the very first model-experiment carried out at the Poona laboratory for the Hardinge bridge. Canal engineers had no objection to splayed guide-banks, which suited the simple problem of passing a meandering river over a canal weir. On the other hand splayed guide-banks were anathema to the railway engineer for reasons too numerous to be detailed here.

The committee of engineers appointed by the Railway Board to examine and report on the training works of the Hardinge bridge had held their first meeting in the unavoidable absence of the official chairman. At that meeting the Committee had laid down on the map two splayed lines connecting with the right and left guide-banks, evidently with the idea of collecting the river as in a funnel and thus passing it through the bridge. On the right bank the line indicated a proposal to extend the Damukdia guide-bank in the direction of Raita, and on the left bank the splayed line passed at the back of the Sara protection-bank and gave it the appearance of a protuberance. The latter line indicated a proposal to substitute a long bank of easy curvature for the Sara protection-bank. No one had observed that if a splayed line were drawn over any converging guide-bank or over any less splayed guide-bank it would make a protuberance of the head of the converging guide-bank, or of the head of the less splayed guide-bank; nor had it been observed that, owing to the mean axis of the river coming in towards Sara from the west, the so-called protuberance became a part of a pair of slightly splayed guide-banks, as shown in Fig. 16, Plate 2.

At the second meeting the Committee had prepared a list of the model-experiments which they wished to have carried out at the Poona laboratory. The experiments contemplated the retention of the Raita protection-bank and the demolition of the Sara key position, although the selection

*** In the absence of the Author, these remarks were read by the Secretary.

of the site below Sara and the future safety of the bridge had depende and did depend, on the possibility of making permanent the positio both at Raita and at Sara. It would be observed that the collection o the river by means of two widely splayed guide-banks, although qui unsuitable for a railway bridge, had become a fixed idea on which th model-experiments were founded. It subsequently had appeared tha with the commencement of the model-experiments, the initiative in th task of remodelling the training works had passed to the Poona laborator where the virtues of converging guide-banks were evidently qui unknown.

It was deplorable to think that some chance preference for splaye guide-banks, combined with a single ill-conceived and misinterprete model-experiment, might have led to the supreme folly of destroying th only natural feature on the left bank between Sarda and Pabna from whic any assistance could be obtained in connexion with the bridging of th Lower Ganges.

Sir Leopold Savile, Vice-President, said that the Author, in sum marizing, amplifying, and bringing up-to-date the work of Mr. J. R. Bel Sir Francis Spring, Mr. Alexander Izat, and others, clearly indicate the difficulties which engineers had experienced in constructing larg bridges in the northern plains of India, and pointed out how the intro duction of what was known as the Bell bund had gone a long way to mak the building of such bridges possible by bringing the cost down to a reason able amount.

He had himself been employed on the Bengal and North Western Rai way during the time that the Gokari-Chalka bridge had been constructe The protection-banks of that bridge had been originally designed ver closely on the lines of the Bell bund, the upstream banks diverging fro the bridge itself, but due largely to a washout on the left guide-ban during construction that bank was reconstructed parallel to the one o the right bank, and he thought that he was right in saying that a subsequent bridges constructed on that railway under the direction o Mr. Izat had the guide-banks parallel to each other and generally norm to the centre line of the bridge. There were three forms of guide-ban layout, namely those expanding towards the bridge, those parallel to eac other, and those contracting towards the bridge. There could be litt doubt that the second type was the best, and it had usually been adopte by Mr. Izat for railway bridges for the Bengal and North Wester Railway.

There was little doubt that the information given in the Paper woul be of the greatest assistance to engineers having to construct bridges ov rivers of the character of those found in northern India, and at the sam time there was little doubt that before deciding on the site, size and pr tection of bridges of any magnitude, the fullest investigation would hav to be made both into the past history of the site of the bridge by boring

aken over the site, and if possible by means of model-experiments, efore any definite scheme could be evolved. One thing which the Paper mphasized, and which was not generally recognized, was that the design nd construction of protection-works for a bridge of that kind was quite s important as, if not more important than, the design of the bridge tself. He would only add that no one without great experience of what ad happened in the past could hope to design satisfactory works of that ature, and it seemed improbable that any formula would be evolved which would indicate the results of such works on rivers of the type met ith in northern India.

Mr. G. J. Griffiths remarked that with regard to the protection of ier-foundations, it might be said that rivers did not like bridges; as soon s a bridge was thrown across a river the river attacked it. The experience f engineers in India suggested that in many cases the river started off o cut through the approaches, rather than to attack the piers and abut- nents. In his experience, in the case of small bridges in Great Britain t was the piers and abutments which were always subject to scour. With egard to the deposit around the piers, it would be of interest to have some uidance in regard to the depth of the stone pitching deposited both at the pper and at the lower end in the first instance.

Another point which interested him was that "one-man" rock blocks vere found to be more stable than larger blocks; he assumed that that vas because of the "liveliness" of the sand, which so far as he could see, ecame almost a quicksand. His experience had been that scour had lways appeared to commence at the toe of an apron, and yet those who ad had experience of Indian rivers did not increase the thickness of the pron at the outward end to the extent which would appear to be necessary o form a sufficient depth of deposit on the slope. Would it be con- idered desirable to increase that thickness, if necessary, or had experience hown that the provision which was allowed for in the taper was sufficient? Ie would have thought that heavier blocks would be more efficient at the utward end than lighter blocks. The sand—owing, he believed, to a me deposit—would apparently stand almost vertical to a height of 0 feet. Was that fairly generally the case, or did it occur only in a very w cases?

He hoped that notice would be taken of the Author's suggestion that rther investigation into the behaviour of sand and other materials nder varying conditions would give results of the greatest usefulness. ven though in Great Britain only small rivers had to be dealt with, the rinciples were the same, and if they were used, even on a small scale, ey would be of great assistance in the future.

Sir James Williamson said that mention had been made by the Author f three of the numerous large bridges on the railways with which Sir James as connected. Until about the beginning of the present century the osi river had been stabilized for probably at least 2 centuries. Prior

to that it had built up the country to the eastward for a distance of som 30 miles, but towards the end of the last century it commenced to brin down unusually large quantities of silt—the earthquake of 1897 bein probably a predisposing cause—and to spill that silt over to the westwar until it had laid waste a tract of country extending almost down to th Ganges. At the beginning of last year it had moved farther westward giving it a westward swing at present of some 40 miles. It had bee possible to salve some 60 miles of railway, but he was afraid that anothe 66 miles would disappear. At the point of bridging on the left bank of th Ganges, however, there were so far no recent silt-deposits, although th whole country was alluvium.

The bridging of the Kosi had been regarded as rather a bold experimen when the late Mr. Alexander Izat had undertaken it towards the end of th last century. The only feasible crossing was at a point which was at tha time about 5 miles upstream from the junction with the Ganges. Th approach, which the Author criticized, had had to be so aligned that it ha an oblique direction towards the river. With the westward swing to whic he had referred the river ran parallel for 35 miles or so with the main lin running along the north of the Ganges, which was only a few miles away The Kosi had been held so far, and at the great embayment which ha occurred above the right guide-bank there had been put in what was calle a Denny groyne, which he thought would hold the river and stave off th development which the Author feared.

The Author also referred to the Bagaha bridge, which had also been buil by Mr. Izat. It involved the crossing of a large river with an unknow catchment-area. Since the time that it was built surveys in Nepal ha shown that the estimated catchment-area was about one-third only of th actual area. At that time, moreover, the great depths to which the rive could scour were unknown. Wells were put down for the foundations o the piers to 90 feet below water-level, but in 1924, when the bridge was i great danger, he had measured depths of over 100 feet.

The river had too steep a slope to bridge in the manner adopted One of the piers had been scoured out, velocities of 17 feet per secon having been measured, and two of the girders had disappeared. Som time later, about $\frac{3}{4}$ mile downstream, the corner of one girder becam visible, whilst 12 years later a sandbank was eroded and the second girde was discovered 6 miles downstream.

With regard to straight guide-banks, he pointed out that, if it wer impossible to complete the full guide-bank with the curved head in on season, it was laid straight with the idea of finishing it off to a curve radius some time later, but it had seldom been possible to do so. Dee channels formed round the end of the guide-bank, which in some case made it almost impossible to continue the construction for the tim being.

In the earlier days aprons for guide-banks had undoubtedly been mad

much too light. The guide-bank had been proportioned according to the estimated depth of scour, but the ferocity of attack and the degree of turbulence had not then been visualized. The attack began under the toe of the apron, and might continue until the apron assumed the appropriate submerged slope and was supposed to be consolidated; at some period, however, and generally about from half to three-quarters flood, the attack went still further, and there was a slip, so that unless the gap formed at the junction of the permanent slope and the apron was filled up at once there would be a breach. A heavier form of construction had therefore been adopted and provided more stone which could trickle down into the gap. In practice aprons had to be continually made up until in time a state of permanency would probably be reached, but none of his own bridges had yet reached a state of permanency. At places in northern India where stone was not available *kunkur* was used, as it tended to become cemented by mud and sediment, with the result that it would drop *en masse* instead of piece by piece as desired.

Another question of interest was the pitching around the piers. As Chairman of the Hardinge Bridge Committee which was formed in 1934 to put forward proposals for that bridge, he would like to make a few remarks. The Government of India at that time were so alarmed by the damage which had occurred at the Hardinge bridge, which was simply due to a sudden sinking of the apron and which led to a very serious breach in the guide bank, and by the vast sums of money that it was estimated would be required to hold the bridge, that they appointed a committee. On the committee were several canal engineers from the Punjab, two of whom were very keen on model-experiments; Sir James, however, having had nothing to do with such experiments, had been rather sceptical about their help. The committee voted a sum of money for those model-experiments, which were carried out at Poona by Mr. C. C. Inglis, M. Inst. C.E. The committee had formed their own theories from experience, and when the model-experiments were carried out Sir James had been astonished to see the accuracy with which they upheld the results of experience in the proposals which had been brought forward.

As would be seen from Figs. 14 to 17, Plate 2, the Ganges in the vicinity of Raita and Sara, where the Hardinge bridge was situated, flowed as a great double curve. In 1908, during the controversy about the site of the bridge, Colonel T. Gracey and Mr. Alexander Izat, who had had great experience of bridging alluvial rivers and of training works on the Lower Ganges, had recorded their opinion that the bridging of the Ganges near Raita, and the subsequent maintenance of the bridge-works, would be too risky and should be avoided. That had indeed proved true.

In 1934 the Hardinge Bridge Committee had come to the conclusion that the Sara spur on the left bank, 3 miles upstream from the bridge, which so greatly aggravated the flow (already disturbed through the sinuosity of the river), and was responsible for deflecting a concentrated

attack towards the right guide-bank, which had been all but destroyed, should be removed. Further, amongst other recommendations, it was concluded that the main defences of the bridge should be restricted to the guide-banks, and as those were in length only about half the waterway of the bridge, it was recommended that they should be lengthened. The breach in the right guide-bank was undoubtedly due to deep undercutting and a sudden dropping of the whole of the apron pitching, and not to the causes stated on p. 191. At Sara, where the regimen was already disturbed by several natural bends, detached fenders or spurs intended to guide the river tended to aggravate matters and do more harm than good.

The alarming depths of scour near the right-abutment piers, where the river guttered through three or four spans, could only be dealt with by pouring in pitching material to maintain a safe limit of hold for the pier foundation-wells. Pitching deposited around a pier was not normally completely washed away from the vicinity of the pier, but through undercutting it settled in a downstream direction until eventually the greatest scour-depth was reached, and there formed a toe or foundation against which subsequent material was held and built up.

In numerous similar situations, for instance in the case of the Kosi and Elgin bridges, as much as 300,000 cubic feet of large and small material had been deposited around single piers to maintain a safe limit of grip, and at the latter bridge, at least, permanency had perhaps not yet been attained.

Dr. Herbert Chatley remarked that there were several rivers in China which compared with the Indian rivers referred to as far as discharge and even slope were concerned, but they had that discharge and that slope in the lower part of their valleys and the beds were of comparatively soft materials, so that it was difficult to draw any comparison. The Yellow river had three bridges across it, which, so far as he knew, were all supported on the southern side on fairly high ground, whilst on the northern side there were approach-viaducts which stretched right across the alluvial plain. These bridges had been placed where the river had not so far attacked either the bridges themselves or the approaches, so that there was no direct comparison possible, although general principles were laid down in the Paper which should apply to such a river.

It seemed to him that the great point about the Bell bund was that it was a measure designed to defend a bridge or any similar structure against the migration of the meanders. There appeared to be a good deal of room for research into the subject of the proportions of meanders and how they varied according to the slope, the material, the velocity of the river and the width of the alluvial valley, if it were a confined valley. In the Paper there were references to definite arc-chord ratios, based on experience and probably perfectly true in the circumstances where they were observed, but probably quite incorrect in other cases. It was very well known, for

example, that the meanders on the Mississippi river had a far greater arc-chord ratio than the figures given in the Paper. He suggested that in a perfectly free alluvial plain there was a tendency for the energy lost at the curves roughly to equal the energy consumed in friction on the bed. That was merely an hypothesis, but if it were so, and if it were possible to determine the loss of head on a meander of given curvature, it would perhaps be possible to deduce some abstract and more general principle relating to the whole behaviour of meanders in all kinds of rivers, which would undoubtedly be of great interest and probably of practical value.

His attention had been drawn to certain rather curious points. One of those was the use of what was a kind of *pierre perdue* in a special form. That method caused uncertainties in the subsidence of the apron, and fractures and slips might occur, causing great expenditure of material, and probably in many cases a failure of the apron due to there being no material at the spot when required. In China, as far as his own experience with smaller rivers went, it was customary always to use mattresses in such cases, and he believed that that had generally been the practice in America. On the Mississippi mattresses had been used to a very great extent, and it would be of interest to consider just where the economic balance lay between the method of an unconfined and free apron which was expected to subside upon erosion, and the definitely designed mattress of brushwood or other material which was laid in place and which, barring extreme accidents, would not break but would subside into any kind of pocket that occurred.

He noticed that all the designs showed a simple circular curve at the head of Bell bunds, whilst great importance was attached to a particular taper on the banks. He was not in a position to discuss whether that taper was of value or not, but he thought that if it were worth while to taper the banks it was also worth while to use a diminishing curvature on the heads, because as the water passed round the curve there was less loss of energy than if the curve were circular. That difference was due to the fact that when water passed round a bend with diminishing curvature a large fraction of the flow was irrotational and recoverable, whereas if the water passed round in a circular curve most of the energy of rotation was lost and was carried on in a swirl, which would sooner or later do damage. He would therefore suggest that the heads of guide-banks should be made of diminishing curvature instead of circular curves which suddenly changed to a tangent. It would cost no more.

Colonel Sir Gordon Hearn said that he proposed to deal mainly with the Hardinge bridge over the Lower Ganges, with which he had been concerned from 1922 to 1926 as Chief Engineer and as Agent for the Eastern Bengal Railway. The bridge replaced a ferry in 1915, and was vital to the Eastern Bengal Railway, as it carried traffic to and from Bengal north of the Ganges, Assam and the Mymensingh district. He had taken

every possible opportunity to make trips up and down the Ganges, and on one of those trips he discovered the possibility of ascertaining the course of the main channel by the formation of islands. He had had the advantage of many talks with the engineer in charge of the bridge, who had had at least 16 years' experience, and when he (Sir Gordon) had had responsibility he had always acted on the "bend and cut-off" principle. He placed complete reliance upon it.

He found that in several of the rivers which had to be crossed there was a tendency among engineers to place spurs near the bridge, but in his opinion that only retarded the embayment, and the proper action to take was to strengthen and possibly to lengthen the guide-bank, and to squeeze the embayment as much as possible until a cut-off was obtained. *Fig. 9* (p. 168) agreed with his views, but it should be recognized that a cut-off might be followed by a swing-over of the serpentine course and an attack upon the other guide-bank.

He thought that the embayment at Sara started before 1925, because a year or two previously a sandbank was disclosed at the bend below Raita at low stage. In 1925 embayment was active, and it was of some interest to mention that a 60-year old tree there was engulfed. That indicated a cycle of more than from 30 to 40 years, as the Author suggested, and gave more time to restore the position. He agreed entirely that Sara was the key position, and that it had if possible to be restored; it was completely lost, he understood, in 1935. The reduction of the stone-reserve was, he believed, based upon his recommendations, but that reduction had been contingent upon the armouring of the back of the Sara bank-head, and he understood that that had not been done. It had not been anticipated that that particular form of attack would take place for some time, and no stone had been pitched there.

He had some doubt about the Author's design of the radius and central angle of the curved head. He thought that it should be of a radius equal to that of the worst possible embayment, and it should extend so far as to provide a reverse curve of from 90 to 120 degrees, so as to give the river an easy transition from the embayment curve, avoiding eddies and consequent scour. He agreed in that respect with the views of Mr. C. C. Inglis.[1]

He had previously drawn a comparison between the Aie River bridge and the Hardinge bridge.[2] He believed that his instruction to curve the

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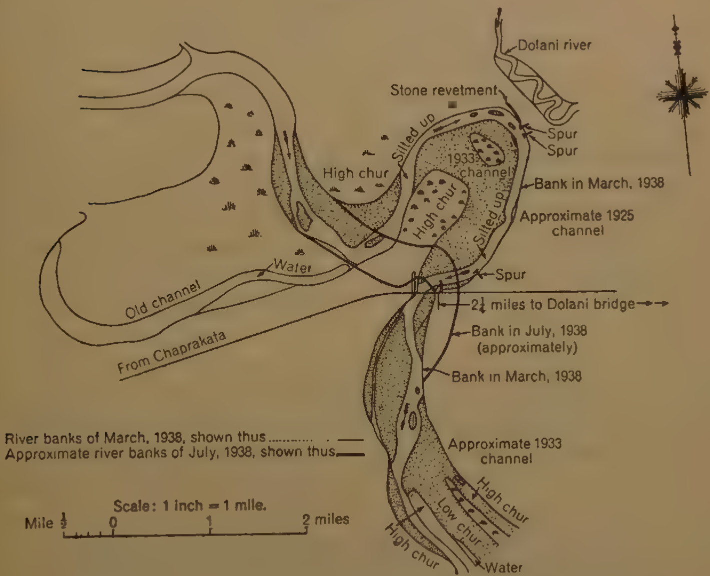
[1] *Correspondence on* the late Mr. B. L. Harvey's Paper, "The Restoration of the Breach in the Right Guide Bank of the Hardinge Bridge." Journal Inst. C.E., vol. 6 (1936–37), p. 331. (October 1937.)

[2] *Discussion on* Mr. J. D. Watson's Paper, "The Reconstruction of the Empress Bridge over the River Sutlej on the North Western Railway, India," and on Col. W. Macrae's Paper, "Training-Works in Connection with the Shortening of the Empress Bridge over the River Sutlej." Minutes of Proceedings Inst. C.E., vol. 237 (1933–34, Part 1), p. 136.

left guide-bank of the Aie bridge had not been adhered to, and the bank had been made straight, as shown in *Fig. 20.* A cut-off in 1933 had been repeated in July 1938 under much worse conditions. The left guide-bank had been scoured away with 400 feet of the bridge approach, and part of the right guide-bank had gone also, cutting the main line to Assam. The design of a guide-bank to withstand such a cut-off required consideration. A curved head might have acted as an arched dam, but a straight bank could not. Apparently the bend cut-off ratio in the Aie was 4. There was a possibility that the alignment of the bridge at an angle

Fig. 20.



Aie River, showing Breach at Mile 247G in July, 1938

to the main axis of the river induced persistent attack on one guide-bank.

He had never considered that the Damukdia bank at the Hardinge bridge was justified. Some sinuosity was bound to be expected with outworks 3 miles above the bridge, but a large embayment was improbable so long as the bridge acted as a sluice in " drawing " the main channel. There had been some anxiety in 1925 because of erosion along the bank near the Paksay colony, but he had rightly foretold that that was temporary. The re-opening of the Damukdia channel should herald a period of comparative peace, as the Author suggested, but the railway engineers did not envisage that. It was proposed to extend the left guide-bank apron to deal with heavy scour down to R.L.60, or 187 feet below high-

flood level, and to extend the bank, which was 2,500 feet long, by 5,500 feet. The late Mr. B. L. Harvey, M. Inst. C.E., in *Fig. 2* of his Paper [1] showed a 1933 flood-level of just above 45 feet above mean sea-level, but in 1934 it was 47 feet above that level. The velocity that year was still 13 feet per second, with a discharge of 1,750,000 cusecs, much less than that for which the bridge had been designed by the Author. He thought that pitching round the piers had been overdone and had reduced the waterway, with consequent scouring. Since then there had been more and more pitching round piers Nos. 4 to 7. He did not understand the term "danger limit." Scouring up- and downstream of the bridge did not involve equal danger. In his experience a scour-slope downstream of 1 in 2 was normal in many bridges on the Sara-Serajganj branch without any danger to the piers, although it affected the guide-banks downstream.

He wished to associate himself in the tribute to the late Mr. J. R. Bell. He (Sir Gordon) thought that the Author dealt rather gently with the recommendations of the Hardinge Bridge Committee of engineers, and perhaps the Author's classification of rivers showed that experience elsewhere was not applicable to the Hardinge bridge. Even the late Sir Francis Spring's Paper [2] was not universally applicable. Sir Gordon did not understand the object of preparing a contoured plan of the whole country, as desired by the Committee, over an area of 8,000 square miles east of the meridian of Lalgola.

Mr. T. A. Curry remarked that in deltaic areas the construction of permanent buildings and embanked railways and roads was to be deprecated as being harmful to agriculture and public health. If a river were allowed to develop and reclaim naturally the country through which it passed, then the country remained healthy, the land was fertile for agriculture and was easily drained, no swamps occurred and no water-borne diseases such as cholera, dysentry, and malaria developed. If, however, such works had to be built, then it was very desirable that sufficient openings should be left in the embankments, that arrangement for drainage should be made, and that the river should be left unfettered as far as possible. Those remarks were prompted by the description of the Kosi river, which moved a distance of from 12 to 14 miles in 7 years.

The Author had discussed the alignment of guide-banks, and was strongly of opinion that guide-banks should converge to a narrow throat above the bridge. It seemed to Mr. Curry, however, to be open to doubt whether the advantages so gained outweighed the disadvantages. The Author stated that if the guide-banks converged to a throat upstream then sandbanks were not formed near that throat, which did happen if the guide-banks

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[1] "The Restoration of the Breach in the Right Guide Bank of the Hardinge Bridge." Journal Inst. C.E., vol. 4 (1936–37), p. 26. (November 1936.)

[2] Footnote 2 (p. 137.)

were splayed. The length of the bridge was, however, usually considerably less than the normal full width of the river, and the channel between the guide-banks could be considered to be partly in the nature of a flume leading water to the bridge. If, therefore, the guide-banks were arranged to, say, a splay of 1 in 20, or even of 1 in 10, then the river would approach the bridge in a streamline manner and with a more uniform flow. At the Hardinge bridge, where as far as he knew the guide-banks were parallel, there had been a considerable accretion of sand on the left-hand side, and except for bays Nos. 2 to 6 navigation was not possible under the bridge, except, perhaps, during the height of the flood. The result was that the flow of the river was concentrated in spans Nos. 2, 3 and 4, and the steamer companies were constantly complaining of the difficulties of navigation. It would appear, therefore, that if the guide-banks were splayed upstream, there would be no formation of sandbanks or accretion of sands near or under the bridge, and the flow would be more uniform and streamline, and navigation would be easier.

The Author had laid down the principles of the design of aprons for guide-banks on pp. 172 to 175. Mr. Curry thought that what occurred was that after the apron had been laid in the initial instance and the river had begun to rise, scour took place underneath and at the outer edge of the apron; thereupon a considerable fall of stone into that scour occurred, and an almost vertical face was left above the scour. Then some stone fell down and partly lined that vertical face above the scour until a fresh scour occurred, and so a succession of scours and falling-in of the stone continued until there was an under-water lining of the side of the river. The thickness of that lining, however, was not uniform, and the only method, as far as he could see, to overcome that difficulty was that an additional supply of stone should be provided so that the lining everywhere should be not less than the minimum thickness.

The Author had recorded his opposition to the recommendations made by the Hardinge Bridge Committee. Mr. Curry had served on that Committee, and he wished to state that the river and the training works had been carefully inspected in 1934 and 1935, and that the records had been closely studied. At that time the Sara guide-bank was acting as a spur, around and below which eddies were scouring the river-bed to great depths, and was directing the main current of the river towards the right guide-bank, along which the flow was turbulent and eddying. Such conditions were prejudicial to the safety of a guide-bank for a bridge. The approach of a river to a bridge should be with as smooth flow as possible, a point that was stressed by the Author on p. 163.

The recommendations of the Committee were intended to provide a smooth and streamlined approach to the bridge, and accordingly a recommendation was included to remove the projection at Sara that was causing the main current to impinge on the right guide-bank. In regard to the possible development of the Sara embayment after the removal of the Sara

promontory, the Sara bank was known to consist of hard clay (a fact mentioned by the Author on p. 195, where it was also mentioned that the Sara bank had stabilized the river for 150 years), whilst the probability of a cut-off opposite Sara was considered. The extent of the area containing that hard clay was not known, and so a recommendation was made to determine it by means of borings at specified intervals. Behind the left bank, near and above Sara, the country sloped towards the east, but no reduced levels of the ground were available. In order, therefore, to deal with the overspill from the river, if such action were necessary, a contour survey of that area was recommended.

With regard to the periodical changes in the course of the river Ganges in the deltaic area (mentioned by the Author) the observations made in recent years at Rampur-Boalia, at the offtake of the Mahtabhanga river above the Hardinge bridge, and at the offtake of the Gorai river below the bridge, seemed to indicate that the Ganges was trying to revert to its 1868 course.

Mr. F. C. Temple said that over a considerable part of one district in India with which he had been concerned an extra 6 inches of afflux meant at the height of the flood the submerging of about 15 square miles more country, and although it might cost the railway a good deal to put in the necessary waterway to avoid that happening, he thought that the saving in damage to the country would pay for the extra waterway in a comparatively short time.

The river Gandak was busily engaged in land-building and was bringing down a great deal of material. It was important, as Mr. Curry had said, that the land should be built up evenly ; if it were built up unevenly, there would be a tendency for the river to go into the low places, and he thought that that was going to happen with the Gandak, and that it was probably going outside the other big bridge which Sir James Williamson had over the river down near its mouth into the Ganges.

The Bagaha bridge was too short. He thought that if three bridges each 1,000 feet long had been put across the area which the river had to spill over there would have been a good chance of their holding, and the land could have been built up evenly. He was surprised that the slope at Bagaha was stated to be only 3·3 feet per mile. The ground-slope about 80 miles further east, where the land was very similar and a good deal further from the foothills, was 2·6 feet per mile, and certainly the appearance of the ground and of the river at Bagaha suggested a much steeper slope than 3·3 feet per mile.

The Paper did not refer to certain factors. There was the tendency of all the rivers in the north of the Gangetic plain to move to a limiting position to the west. The Ganges appeared to have gone as far west as it was going. There was also a tendency for all the rivers to go as far south as possible before turning east. The Ganges could not go any further south than Allahabad because it was pinned by a rocky bastion 500 miles away at the

north-eastern corner of the Central India rocky plateau. The strata on the two sides of the river were quite different; on the right bank the strata were continuous and were wide and deep, much deeper than the annual scour of the river, but on the left bank they were patchy, short, and shallow. The hard masses on the south, as at Allahabad, Patna, and many other places, were reliable, but the hard masses on the north bank were quite unreliable. There should be a very good chance of holding the river at Allahabad because it appeared to have finished its movement at that point, and if that were so it ought not to be difficult to hold a permanent bridge there; the holding of the bridge at that point would probably be all to the good of the country, because the Ganges had almost ceased to build land in that area. The Kosi was a much bigger river than the Gandak, but by the time it reached the bridge it had flattened out and the plain was wider, and the flow was less swift. In his opinion it was the business of the Kosi to continue building up land right across the width between its banks, which were 60 miles apart and not very high. The important thing was that it had to build up the land evenly, and for that reason it was wrong from the point of view of the country to try to hold the river to one bridge in one place. Assuming that three bridges were built, any one of which could contain the river, then if the methods indicated in the Paper were used to train the river first to one bridge and then to another, the land could probably be built up evenly, and in the future that was probably one of the best things to which the principles of river-training could be applied.

The ultimate failure of the Sara clay was another instance of the untrustworthiness of the hard masses on the left side of the river, but the Ganges had stayed at Sara for a very long time and was discharging on the east of the delta. Sooner or later in the building of the delta it had to go back to the west, however, and it would therefore be desirable to have a considerable spill down on the west long before it actually wanted to move, so as to avoid Calcutta being washed away!

*** **Lieutenant-Colonel William Macrae** observed that the Paper gave what to his mind was the first reasonable and acceptable description of the main failure of the Hardinge Bridge right-bank main Bell bund, and of its causes. The Author made it clear that the failure was due to wave-action and surging sucking up the very fine silt underlying the inner section of the apron, and causing wholesale slipping of the slope, and was not due to any of the extraordinary assumed causes on which the Committee's proposals (pp. 190–191) appeared to have been based such as "the protuberance at Sara", which had no existence on the ground.

While not able to go quite as far as the Author in his conclusions and proposals, he was convinced that the first steps to be taken at the river-training works of the Hardinge bridge were the strengthening of the armour

*** This contribution was submitted in writing.—SEC. INST. C.E.

of all slopes and of all inner sections of aprons to prevent sucking-out o bank-material by wave-action and surging, the restoration, as far as cir cumstances permitted (pp. 196–197), of the Sara key position of th left-bank defences, and the extensions of aprons to meet the depths o scour that exceeded those against which the original defences wer designed.

The sucking-up of fine material underlying the inner part of an apro was a process of which he had had no experience, but after the failure of a Bell bund at the Kalabagh bridge on the Indus, where the behaviour of th river was more akin to that of a mountain torrent than to a river in alluvia plains, he had noted that blocks of " one-man " rock had been sucked u from the inner apron and deposited downstream ; although not the caus of that failure, their evidence had proved the unsuitability of the site for a Bell bund and the need to extend the bridge to the permanent bank, a had been done.

The description of the failure of the river-training works of the Bagah bridge was of interest as it brought out clearly the unsuitability of the rive at that site for training by Bell bunds. He understood from Sir Jame Williamson that large concrete blocks had been tried to save the pier, bu they had failed ; apparently they had behaved exactly as described b the Author in comparing their behaviour with " one-man " rock, the san scouring out from below them, and allowing them to sink quickly and s become ineffective. Similar results had apparently followed from a tria of large blocks round piers of the Hardinge bridge, upstream of a deep scour It was not unusual for engineers to suggest the use of such blocks, and th Author had shown clearly why they failed in such conditions.

In regard to the Kosi bridge, he agreed with the Author's opinion o p. 150 that nothing could save the railway from being breached to the east ward if the Kosi swung sufficiently far in that direction. The Kosi Bridg river-training works were the one stationary factor in the course of the river and it was a matter of good fortune that they had not been outflanked. Th probability that that would happen might induce the Government of Indi to construct a Ganges bridge either at Bhagalpur Ghat, at Mokameh Ghat or on the stretch between them, and in such case it was to be hoped tha the engineers would give due weight to the principles for class C river laid down in the Paper. The construction of a bridge on that section ha been advocated in the past, but had been barred on financial considerations Its desirability was obvious and non-political, and consideration of th Author's opinion might lend sufficient weight to enable advocation of it construction to overcome financial difficulties.

The Author stated on p. 162 that as soon as pier-wells were sunk to ful depth they should be surrounded by an apron of pitching stone, but n reasons were given for that dictum. At one time it was impossible to sin such piers as deep as was desirable, and such pier-aprons were essential With modern appliances the depth-limit was more a question of financ

han of practical engineering. If the maximum pier-scour of an unprotected pier could be estimated and a pier could be sunk to a depth suitable to withstand that scour, the heavy recurring cost of maintaining such aprons would be eliminated as a counter-balance to the extra capital cost of those piers, while the extra length of costly next-to-land spans shown by the Author to be needed to allow for those aprons would also be reduced or be rendered unnecessary. In the class A rivers of the Punjab, where the sand was not fine and silted as in Bengal, and where an allowance of 40 feet below low-water level for maximum bund scour on the straight had been found to meet requirements, he would be satisfied to do without pier-aprons around piers sunk about 150 feet below low-water level to allow for 80 feet of pier-scour.

On p. 176 the Author inferred that the Indus had analogous characteristics to those of the Ganges, and probably to those of the Brahmaputra. Lieutenant-Colonel Macrae believed his surmise to be correct in the case of the Brahmaputra; in the case of the Indus, however, only the soil-characteristics, gradually getting finer down the river, were similar. On the other points the Indus characteristics differed from those of the Ganges owing to the plains of the Punjab and Sind being outside the regular monsoon-area.

The rise of the Punjab rivers was less steady than that of the Ganges, being due to melting snows and rain-storms on the hills; floods and the high-river period lasted a shorter time, permitting Bell bund aprons to be laid complete in one season at or within a foot or two of low-water level, and the maximum discharge decreased at each measurement-point downstream until one river was joined by another, instead of increasing as was the case of rivers in monsoon-areas. The records of the Punjab Irrigation Department, Discharge Branch, and of the North Western Railway, showed that clearly.

On the Indus those maxima decreased markedly below Kalabagh until the Panjnad reinforced the river with all the five waters of the Punjab, and from that point maxima again decreased as floods travelled down past Sukkur and Kotri to the sea.

He was not in possession of copies of the records that were available at Lahore (N.W.R.) and elsewhere, but he doubted if the maximum discharge of the Indus at and below Kotri brought it under the Author's definition of class C rivers. In fact, the Punjab rivers, with some reservations for the Jhelum, seasonally became "dying" rivers and behaved as shown in *Figs. 8* (*b*) (p. 168), from which it could be seen that a natural development was for the main current to work downstream to round the left bund nose in the way shown in *Figs. 8* (*a*), in which case the loop attacked, and, if not resisted, cut through the approach-bank. The most marked instance that Lieutenant-Colonel Macrae had seen was at the N.W.R. bridge near Saharanpur over the Jumna, which was never really a "dying" river, but in that case shortness of the Bell bunds and a decided fall of ground to the

east (left bank) had been contributing causes; the Bell bunds had bee lengthened considerably, an artificial cut made across the consolidate jungle-grown strip at the neck of the loop, and the river straightened Records and plans showed that there had been similar behaviour elsewhere and that at some canal-weirs, subsidiary Bell bunds had been mad upstream to deal with those attacks, those bunds being splayed out wards.

It could be said that there were no real permanent banks at the railwa bridges over the Punjab rivers (excluding the Indus), except the right ban at the Beas River bridge at Beas and at the Jhelum River bridge at Jhelum and that those five rivers (Jhelum, Chenab, Ravi, Beas, and Sutlej) ha wandered all over the country. For that reason he thought that i might often be desirable when bridging those rivers to acquire land fo subsidiary Bell bunds (with approach-banks to them) upstream of th main bunds, and in spite of the objections raised to splayed-out bunds b the Author—with which Lieutenant-Colonel Macrae agreed—to site then at an angle of from 30 to 45 degrees splayed outwards, or even to curv them from 30 to 45 degrees, the approach-bunds at right angles to then thus being aligned to join the main bridge approach-banks. At some weir those subsidiary bunds were of "hockey-stick" shape, and functione efficiently, the unarmoured approach-bank corresponding to the stick handle. The distance upstream could be calculated from the norma curve for an active river shown in *Figs. 8* (*a*) (p. 168), as the existence of a upstream Bell bund would prevent the development shown in *Figs 8* (*b*).

Such upstream subsidiary bunds postulated main Bell bunds wit sufficiently curved heads; in the Punjab it could be said that the ol controversy between straight heads and curved heads had been decided b experience in favour of the latter.

On the other hand, his opinion of the soil on at least parts of the Sukkur-Kotri stretch was that it was even worse than that of the Ganges at th Hardinge bridge, melting away when touched by water. That favoure bank-slopes of 1 in 3, instead of 1 in 2. He had not heard that the Bomba Irrigation Department had yet overcome their difficulties with that typ of soil, and the resultant flooding by seepage and destruction of countr along some lengths of the Lloyd Barrage canals.

The Author had emphasized the weakness of the tapering-apron desig at and near the foot of the slope, to which Lieutenant-Colonel Macra would add extra liability to surging and sucking action in the hollow; th Author's proposal for a berm was more suitable for class C rivers wit their large rise and fall than would be the arrangements proposed b Lieutenant-Colonel Macrae for the class A rivers of the Punjab.[1]

Although he knew from experience that the Author was correct in hi

[1] Footnote (1), p. 140.

statement that "one-man" rock did not roll on a 1-in-2 slope, he was doubtful about 7 inches of ballast stopping on such a slope, nor was he satisfied that a case had been made out for the provision of that with 3 feet 6 inches thickness of pitching, except for class C rivers where the bed-soil was very fine with silt. In the class A rivers of the Punjab it could not be said that there had been slope-failures; in the earlier cases the thickness of "one-man" rock was usually 2 feet, and in later cases 3 feet, both without the proposed ballast, but often with some quarry debris.

Whilst he agreed that more care should be taken than had been the case in the past to see that interstices in slope-protection afforded little chance for bank-soil to be sucked out by waves and surging, and that steps had to be taken to ensure no slipping of slope-protection, he saw no need for thick protection on slopes, and he believed that thin coverings, such as bricks and heavy tiles strung on wires as mattresses, or a thin layer of pitching stone with interstices filled with weak cement mortar, could give efficient results, choice depending on distance from quarries and on prices. Whilst he realized the need for revised standards of thickness of inner aprons to prevent sucking-out of underlying material, he was not persuaded that a case had been made out for quite such a great thickness of apron at the inner side, nor such a comparative thinness at the outer edge for all classes, especially for class A rivers in Punjab soil. Shortness of flood-periods produced local variations in strata and meant comparatively short periods of attack by wave-action on the inner sides of aprons, with local attacks on outer edges, which resulted in pitching stone fanning-out locally when dropping, and producing weak patches. It was for the latter reason that he liked to add to the calculated apron-thickness about 1 per cent. per foot of estimated fall for scour. On curves and at noses additions for fanning-out were required. On the other hand, he acknowledged that more time than had been available was needed to study the proposed new standards.

The Author suggested as standard the length of bund tails proposed by the late Mr. J. R. Bell in 1890, at a time when there was not much experience of the regimen of Bell bunds. The Author gave no reasons for that proposal. In Lieutenant-Colonel Macrae's Paper[1] he gave the reasons why he considered that dimension to be excessive and extravagant. He was unable to find any record of attack on bund tails nor of eddies at tails that Mr. Bell thought might develop. Pier-whirls died away in a short distance.

Length upstream depended admittedly on the clear span of the bridge and on the distance from the bridge to the permanent banks, but the absence of the latter made the former a doubtful guide. In the Punjab, where the total span (L) for many bridges was about 2,000 feet, experience had shown that trouble developed in most cases where the upstream length of bund was less than about 2,500 feet.

He concurred with the Author in regard to the desirability of Bell

[1] Footnote (1), p. 140.

bunds converging to some extent upstream, if the river-alignment permitt that at the time of construction; the factors determining the amount convergence were the waterway occupied by the piers and their apro plus their friction and disturbing effects on flow. As a rough guide apron and disturbance-factors might be allowed for in a class A Punj river by adding 50 per cent. to the pier-dimensions. For example, at t Empress bridge over the Sutlej there were seven piers 15 feet 6 inches wi which could be taken as occupying the equivalent of 163 feet of waterwa the upstream length of straight of the Bell bund was 1,450 feet, and t desirable convergence would work out at 81 in 1,450, or 1 in 18 (whi could not have been given at the time of construction owing to the cou of the stream). In a bridge such as that over the Ravi at Lahore, howev with 100-foot spans in contrast to the 250-foot spans of the Empress brid the desirable convergence would be about double that inclination, a would be even more unlikely to be obtainable.

He agreed with the Author regarding the desirability of revision standards for the guidance of engineers in river-training works in alluv country, especially with regard to class C rivers and in very fine soil wi silt.

In regard to the overall-apron design, a form of that was used by instructions to repel a sudden attack by the Jhelum on the left-ba abutment of the bridge near Malakwal. A right-bank Bell bund exist and the stream had always run on that side, but it suddenly switched acr to the left bank. The left abutment was on an unprotected bank high than all but exceptional floods, which bank prior to the attack filled practically the whole of the first span. A small bund was made, rais above highest flood-level and set back from the edge of the river-bank give as wide an apron as obtainable, its alignment being made to cor spond to the right-bank Bell bund; the slope of the bund and the t and slope of the river-bank were hurriedly pitched, the nose-aprons of t head and tail being laid on the high ground, and that foiled the attac Completion as a left-bank main Bell bund was done after the floods h subsided.

Where a head or tail was in high bank he recommended that the ba should not be cut away behind the line of the bund-slope, the nose-apr behind that line being laid on the overall-apron principle. At tails had not found that aprons dropped beyond that line by more than a ve small angle, and at heads he considered cutting away the bank likely to h embayment behind the nose.

There was much to be said for avoiding cutting the river-bank down low-water level behind the front slope-line of the straight length of a bun and using the overall-apron design on the curved portions as well as at t backs of noses where the curves ran into high bank.

He had plans prepared on that principle for the proposed bridge o the Ravi on a chord from Raewind, one of the many projects dropped wh

he financial crisis came, but one of the very few of those on the N.W.R. hat would, he thought, be of great advantage, as it would not only by-pass ahore with its liability to congestion, but would also provide an alternative oute to the North in case of trouble from floods or earthquakes to the Ravi ridge there.

That overall-apron design allowed excavation so that the top of the pron would be level with the existing ground, which at one bank was ractically at high-water-level, providing sufficient earth for a bund with dequate freeboard. Parts of the apron were necessarily a combination f normal and overall-apron design, but there was no apparent objection o that, and in that case the advantages included lessening of the earth-vorks and their cost. To the best of his recollection the former design had llowed what he considered to be an insufficient curve to the heads of the unds because of objection to cutting into the high bank, and also an nsufficient length upstream. The former fault was remedied, and the bjection was met by the overall-apron design; the extra cost of the dditional length upstream was met by shortening the lengths of the ails.

The Author, in reply, stated that, with regard to the inquiry as to the epth of pitching stone to be placed around piers in the first instance, suitable depth would be the apron-thickness given in Table II (p. 175), or the body and tail of the guide-bank according to the class of river for vhich the information was required. The interesting questions of Mr. riffiths were rather difficult to answer, as applying to the whole of orthern India, on account of the extraordinary differences in the con-itions in different parts of the country. The Author had found that eavy concrete blocks sank in and disappeared at the Hardinge bridge nd that specially heavy pitching stone appeared to have no advantage ver "one-man rock," probably owing to the sand being sucked out more asily through the larger voids in the former. The question of the value f the tapered apron as compared with the normal or uniform-thickness pron had been exhaustively considered in the Paper, although the final rgument had not been adduced, that as the development of the apron vas automatic and that pitching stone, when disturbed, went down and ot up, it was preferable to have any excess over the minimum thickness laced at the inner and not at the outer part of the apron in cross section. n his Technical Paper, the late Sir Francis Spring had stated "that he new by many soundings that practically vertical cliffs of pure sand were o be found under water standing 20, 30, or more feet high." The Author ad not been able to obtain any confirmation of that in connexion with the ower Ganges, but from the way in which slips would occur and continue o take place at the same spot for the space of from $\frac{1}{4}$ to $\frac{1}{2}$ an hour, there vould seem to be no reason to doubt the statement. That phenomenon ad first been observed by the Author on the river Chenab at Sher Shah, early 50 years ago.

The information given by Sir James Williamson about the latest mov ment of the river Kosi was most valuable and would be more so if it cou be amplified, perhaps in the Correspondence on the Paper; the figures f miles of railway involved would seem to indicate a change of course fro near the hills and not merely a local movement by lateral erosion. As many of the guide-bank bridges had been built to replace ferries it w common to find that the siting of such bridges had been injuriously affect by the existence for considerable distances of railway alignments suitab only for the ferries.

The Author was surprised to hear that Colonel T. Gracey and M Alexander Izat had recorded their opinion that the bridging of the Gang near Raita would be too risky and should be avoided; they possibly h been unaware of the existence of the resistant clay formations at Rai and at Sara, or had not realized their significance in relation to the proble of bridging the Ganges. They were, however, known to have been desiro of having the bridge built on a site at which, on one bank, there was such permanent or semi-permanent support available. But possibly wh they said near Raita they meant near Raita and not near Sara, in whi case the Author had already recorded an opinion in the same sense.

Coming now to Sir James's defence of the Committee's decision condemn the Sara key position, the Author had met with the difficul that Sir James appeared to have used the term "spur" in other than t sense in which it was commonly employed in hydraulic engineerin There had been a spur condition at Sara which had come about in t course of maintenance and had been the cause of all the trouble, whi culminated in the breach of the right guide-bank by surge wave actio Meanwhile the spur had been removed under the Author's advice and wi the assistance of the river, and smooth flow had been restored in connexi with a curving back of the Sara protection bank, a training work usual described by the Poona laboratory as the "retired guide-bank at Sara Smooth flow had been restored during the freshet which caused the breac the freshet having been the cause both of the breach and of the restorati of smooth flow. At the time of the Committee's appointment there h been no spur at Sara but, by the time of their first visit, there had been perfectly good streamlined stone-pitched bank protecting the Sara k position. It was that streamlined stone-pitched protection bank whi the Committee "came to the conclusion" should be removed and t Chairman had now denounced as a detached fender which did more ha than good. With some inconsistency they had, however, recommend the retention of the detached fender at Raita. They had also come to t conclusion that the flanking guide-banks were too short and that th should be lengthened to the width of the waterway described elsewhere the "correct" length. In coming to those conclusions it would be observ that the Committee had merely reverted to the pair of flanking guide-ban of length equal to the width of waterway, without "detached fenders

which Bell had recommended for the five rivers of the Punjab 50 years ago. They had come to that decision regardless of the fact that the Ganges at Sara was 3 or 4 times the size of the biggest of the Punjab rivers, and that its high floods lasted months instead of days, and in spite of the fact that the banks of the Ganges were higher than the surrounding country and that the sand of the banks and bed of the Ganges was as unstable as any sand in the world. The proposals of the Committee, moreover, entailed a retirement by from 1½ to 2 miles of the whole of the approach-bank from the bridge to Abdulpur, including a dam with sluices crossing the spill between Ishurdi and Abdulpur. The Author's proposals had been set forth in the Paper.

Sir James had been sceptical at first about model-experiments, but in the end was astonished to see the accuracy with which they upheld the result of experience in the Committee's proposals. The Author understood that to mean that Sir James had been satisfied that the first model-experiment described and illustrated in Part II of the Report had proved that the breach in the right guide-bank was due primarily to the "retired guide-bank at Sara," secondly to the Damukdia guide-bank, and finally to a deep-seated slip caused by scour at the site of the breach in the right guide-bank, and that those were the grounds on which it was asserted that "the breach was undoubtedly due to deep undercutting."

The Author stated that the sequence of events which actually brought about the breach would first be described and thereafter the model-experiment would be critically examined.

In 1919–20 cutting had been observed at the Raita and Sara protection-bank and by 1922–23 "the river developed an attack at Sara and nasty eddies were noticed right up against the apron, which fell in at several places." If at that stage the situation had been brought to the notice of the Author, or if a "retired guide-bank" or a Curzon head had been constructed from the upstream end of the Sara protection-bank, it was safe to say the history of the Hardinge bridge would have been very different. However, the erosion of the clay bank above Sara continued and it was not until the 23rd June, 1932, some 10 years later, that the state of affairs had been reported. By that time the damage had been done and a spur-condition had been established at the upstream end of the Sara protection-bank. The spur and eddies were driving the main stream abruptly across to the right bank. The right guide-bank was not long enough for that unexpected development, and as by that time the river was attacking the head, it was no longer possible to extend it. The Damukdia guide-bank therefore came into existence and steps had been taken to restore smooth flow at Sara. During the floods of 1933, however, the main stream continued to be diverted more and more abruptly across, until by the middle of August, 1933, the main stream was impinging on the apron of the newly-constructed guide-bank at about the middle of its

length. During the 10 years that had elapsed since the first appearanc of eddies at Sara, a *chur* or sandbank had arisen along the left bank fron Sara to below the bridge. As the main stream traversed across to the righ bank, the sandbank had extended behind it in such a manner that it greatest width was opposite the head of the right guide-bank, where th channel was consequently much constricted. The breach was caused i the manner described below. From the middle of September the rive had fallen rapidly, until the 25th September. As the river fell the slowin of the current in the shallowing water had led to a rapid deposit of san and silt and upward growth of the sandbank at the already constricte channel. On the 25th September the freshet, bringing a still more rapi rise, had reached the now further constricted channel at the guide-ban head. As the river rose, heading-up resulted, accompanied by surge waves at 2-minute intervals. In the early morning of the 26th Septembe 1933, the water-level being well above the level of the top of the apron those surge-waves, in passing along the face of the guide-bank, had dis turbed the pitching and quarry refuse and, the alluvial clay offering n resistance, had dragged down the sand and silt of which the bank wa composed and caused the stone pitching to drop and expose the sand an silt core, which was rapidly swept away by succeeding waves. The surge wave condition had lasted about 12 hours. The total rise of the river wa about 4 feet, and on the 5th October it had begun to fall. The breach an its consequences had been fully described in the late Mr. B. L. Harvey Paper [1].

The course of the river between Sara and the right guide-bank at th time of the breach ran from Sara to the middle of the Damukdia guid bank, and thence to the head of the right guide-bank. It had taken th river about 10 years to excavate that channel under compulsion of th spur-condition at Sara, and the quantity of material excavated would b beyond computation. During the freshet-rise it was reported that th spur at Sara had been sufficiently eroded by the river to bring about th restoration of smooth flow, and at the top of the rise the surface-wat of the river ran down more directly towards the bridge, but in a few da it returned to the channel at the Damukdia guide-bank, and it continu to take that course until it began to be pushed back towards the left ba by water coming down the Damukdia cut-off during the floods of rece years. It would be noticed that the spur-condition at Sara was an acti agent in driving the river across, but the restoration of smooth flow w merely passive or " permissive " with regard to restoration of the ma river channel to its former course along the left bank from Sara downwar Similarly, the cutting away of the " retired guide-bank " at Sara w merely " permissive " with regard to movement of the river chann and, as stated above, the movement of the channel towards the left ba

[1] Footnote (1), p. 151.

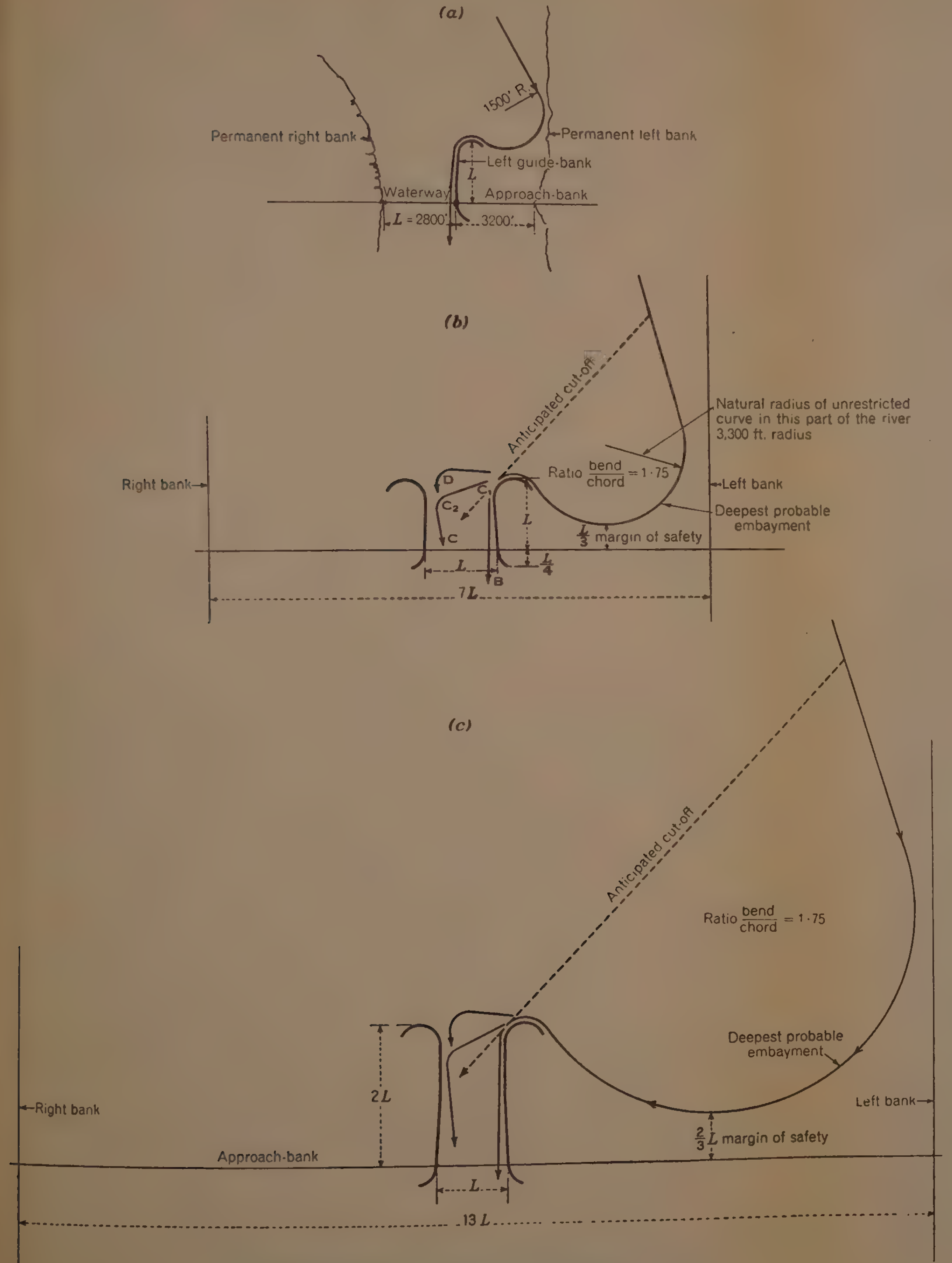
was directly attributable to the driving power of the Damukdia cut-off during the floods.

He would next examine critically the model-experiment referred to above. The breach occurred on the 26th September, 1933. It was stated n the report on the model-experiment that the "local silt was laid" accord- ng to the contour plan of the river Ganges in November 1934. The annual loods might be said to rise in June, attain their highest point in August, and to have arrived at a condition of steady subsidence by the end of October. After the breach, a scheme of temporary repairs, which, however, eft the breach open, had been carried out by June 1934. The works had tood the floods of 1934 remarkably well, but the conditions had led to an extraordinarily deep scour of 187 feet below low-water level, at about 600 eet out from the centre-line of the guide-bank, opposite the open breach. The experiment was carried out with the breach closed and it thus provided " verification " test in which the reliability of the results would be shown y the extent to which the silting-up of the deep scour agreed with the ilting-up which recurred in the actual river. No information on that oint, however, had been furnished, nor was it possible to ascertain what had taken place from the cross sections. It would therefore appear hat the experiment had not "verified". Nevertheless, he would con- ider the grounds on which the theory was based of a deep-seated slip aused by deep scour. It was stated in the Report that the velocities were very high and that the current had scoured silt from underneath he right guide-bank, undermining it by the equivalent of 250 feet from hainage 10 to 25, and that that had occurred owing to the pitching having een laid in material which would not slip with the exaggerated vertical cale. From that and from the cross sections it appeared that the pitching tood like a pent roof and did not rest on the silt. On section 3 at the entre of the repaired breach, with the local silt laid to the deep scour which id not eventuate until a year later than the occurrence of the breach, it ould not have been surprising if the loosely laid local silt, shown in red ash, had run out to an actual slope of about 1 in 50 on being immersed n water, without any assistance from the velocity or direction of the urrent. The experiment, however, if intended to be used to demonstrate depth of scour, had been completely invalidated by the silt opposite e site of the breach having been laid at a greater depth than had been ached in the actual river before the floods during which the breach had ccurred. Nevertheless, leaving aside all considerations previously dduced, there was one respect in which the experiment failed to reproduce e actual conditions for the very good reason that it had not been known the time of the experiment that the foot of the apron slope had already tered a highly resistant formation by about 20 feet. That formation, ferred to as " clay patch " strata, which differed widely from the easily oded Ganges river-bed sand, had made it impossible that any such deep our as alleged could have occurred. Those considerations would seem

to be sufficient to exonerate completely both the Damukdia guide-ban and the "retired guide-bank" at Sara, and to absolve them from an complicity with regard to the breach.

*** The Correspondence on the foregoing Paper will be published i the Institution Journal for October 1939; the Author, in his reply theret will deal further with certain points raised in the Discussion.—SE INST. C.E.

FIGS: 2.



THE PROTECTION OF BRIDGE-APPROACHES BY GUIDE-BANKS OF DIFFERENT LENGTHS.

FIGS: 10.

(a)

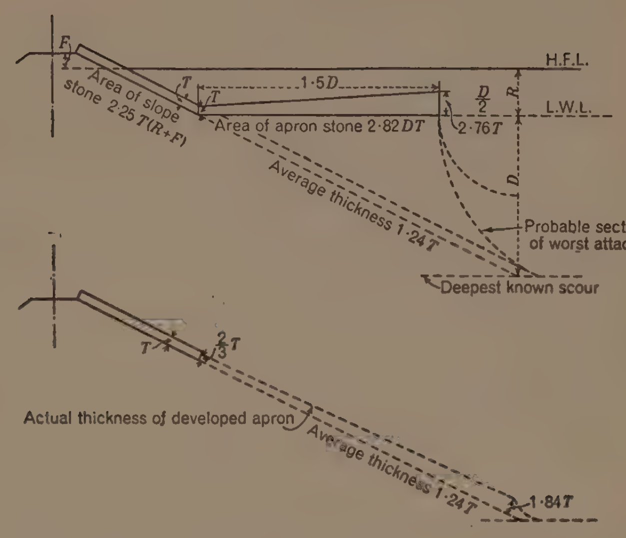


DIAGRAM OF TAPERED APRON.

(b)

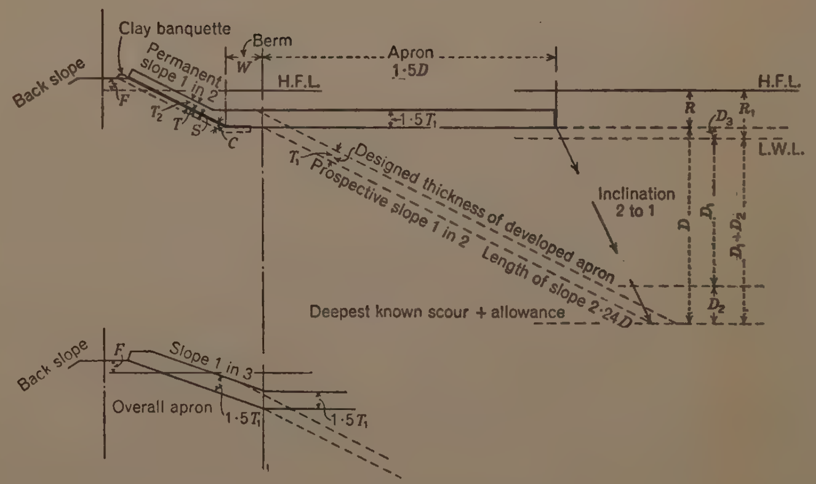
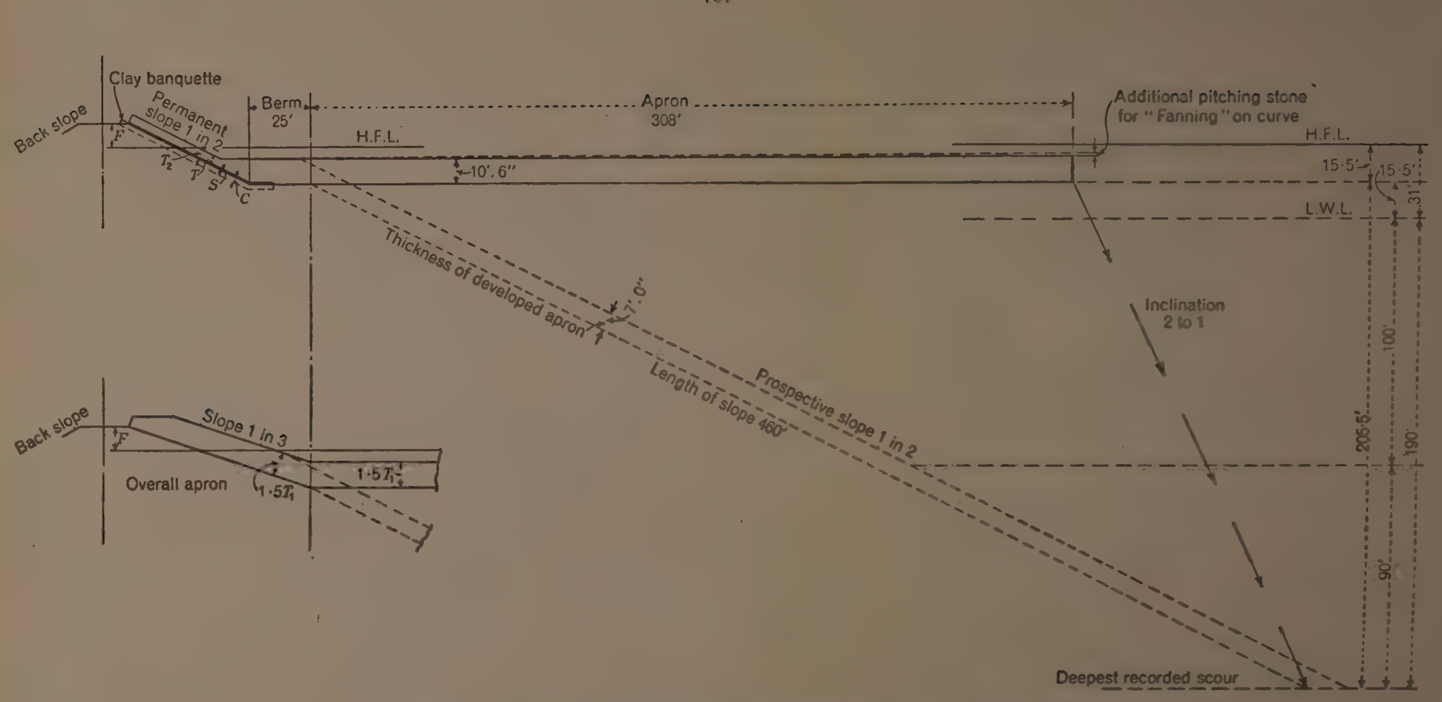


DIAGRAM OF NORMAL APRON WITH ALTERNATIVE SLOPE-COVERING.

(c)



SECTION OF HEAD OF GUIDE-BANK FOR HARDINGE BRIDGE IN SAND AS DEVELOPED FROM NORMAL-APRON DIAGRAM, WITH ALTERNATIVE SLOPE-COVERING

Explanatory Notes.

F = Freeboard
R_1 = Rise of flood
R = Rise of flood above bottom of apron
D_1 = Deepest smooth-flow scour below L.W.L.
(a) observed at cutting bend in soft bank + 33%, or } = "deepest known scour"
(b) observed at hard bank exposed to bend attack }
D_2 = Percentage addition to D_1 for contingencies
(a) including narrowing of river, for body and tail of guide-bank: 25% for Class A, 32% for Class B, and 45% for Class C rivers
(b) including exposed position, for head of guide-bank: 50% for Class A, 63% for Class B, and 90% for Class C rivers
D_3 = Height of bottom of apron above L.W.L.
D = Depth of water for calculation of apron stone
T = Thickness of permanent slope stone
S = Thickness of soling
T_2 = Thickness of covering = $T+S$
C = Thickness of clay covering
T_1 = Thickness of stone on prospective slope below bottom of apron
W = Width of berm: 15' for Class A, 20' for Class B & 25' for Class C rivers
Area of permanent slope stone = $2{\cdot}24\,(R+F)\,3{\cdot}5'$
Area of prospective slope stone = $2{\cdot}24\,D \times T_1$
Area of berm stone = $W \times 1{\cdot}5\,T_1$
Width of apron = $1{\cdot}5\,D$
Thickness of apron = $1{\cdot}5\,T_1$
Area of apron stone = $1{\cdot}5\,D \times 1{\cdot}5\,T_1$
In construction, abrupt changes in the width of the apron should be avoided
Back slopes to be suitably protected by stone pitching or grass

SLOPE-COVERING AND APRON FOR GUIDE-BANKS FORMED OF SAND.

JOINT MEETING WITH THE INSTITUTION OF STRUCTURAL ENGINEERS.

22 November, 1938.

WILLIAM JAMES EAMES BINNIE, M.A., President Inst. C.E., in the Chair, supported by

PERCY JOHN BLACK, Vice-President Inst. Struct. E.

"Some Examples of the Design and Construction of Bridges in Denmark."

By Professor ANKER ENGELUND, M. Ing. F.

(*Abridged Report.*)[1]

THE RAILROAD AND HIGHWAY BRIDGE ACROSS LITTLE BELT.

ᴛ the bridge-site, the Little Belt is 2,700 feet wide, the water being 65 and 30 feet deep near the shores and in mid-channel respectively. The sea-ed is a blue-green clay of very dense and homogeneous composition. It as decided to construct a high-level five-span cantilever bridge with four eep-water piers, using caissons of a special type to enable the work to be arried out in the dry and without the use of air-pressure. These caissons, uilt entirely of reinforced concrete, were constructed with a continuous ng of steel cylinders in the outer wall. Each caisson, approximately 45 feet long by 73 feet wide and 50 to 60 feet high, was cast on land in an verted position. When launched it weighed 7,000 tons, and was reversed nd built up in shallow water. The caisson was then floated out to its nal position, let down on to the sea bed and sunk by boring inside the tube its outer wall. When it had been lowered about 23 feet into the clay, e tubes were sealed by underwater concreting, the caisson was emptied, nd excavation proceeded in the dry. The caisson was then built up to a eight of about 120 feet, on top of which the bridge pier proper was built. he average time taken for constructing one pier, including all caisson work, as 2 years.

The bridge carries a double-track railway, two highway lanes, and a ootway. The distance from centre to centre of the trusses is 54 feet and

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[1] Notes on the Störstrom bridge are not included, as a Paper on the construction f the Störstrom bridge will be read before The Institution in the Second half of the ession. A fuller Report of the Lecture will be found in *The Structural Engineer* for anuary, 1939.

the height of the trusses varies from 51 to 78 feet. The centre span of t
bridge is 722 feet, the two end spans on each side being 542 feet and 451 fe
respectively. The bearings on the two central piers are fixed, all other pi
bearings being movable, and expansion is taken up by one of the links in t
centre span and also at the end of the bridge. The steel used in the stru
ture was Krupp *Baustahl*, with a minimum ultimate tensile strength
77,000 lb. per square inch and a yield-point of 51,000 lb. per square inch.

A Special Method for the Construction of Bridge Piers adopted in Danish Fjords.

In Danish fjords soft layers of mud (" cardium ") up to 100 feet
thickness often occur. Below this may be found additional soft stra
while the depth of water above is seldom more than about 50 feet, but ma
rise to about 80 feet. The " cardium " mud, although quite unable
support any vertical load, has enough cohesion and friction horizontal
to prevent a pile from bending out of its axis. This fact is of importan
in the foundation method described below:

At the pier site, the first few yards of mud are removed and replaced
a sand fill or " cushion " ; a group of piles—usually timber, or holl
reinforced concrete when especially long piles are required—is then driv
with the length of the piles so chosen that their heads are left protrudi
from 3 to 6 feet above the sand " cushion." The piles are driven by
pile-driver mounted on an anchored barge, the heavy adjustable lea
guiding pile and hammer being extended down to the sand " cushion
The piles—most of them raking—are driven either by a pile-extension
by an underwater hammer.

On top of the completed pile foundation is sunk a reinforced-concre
caisson with buoyancy compartments in the outer walls. After a suitab
amount of underwater concreting, the caisson can be emptied of water (
compressed air if the conditions are bad), enabling the concreting to pr
ceed in the dry. Finally, additional sand-fill covered with stone is plac
around the base of the pier. The considerable weight of the pier structu
together with the superimposed vertical load, cause high compressi
stresses in the piles, so that it is difficult ever to introduce more th
negligible tensile stresses in the piles themselves. Attention is paid
obtaining a good connexion between the caisson and the concrete in t
working chamber and between the concrete and the piles, to the prop
grouping of the piles, and to test loading.

A vote of thanks to the Lecturer was proposed by Mr. Ralph Freema
Member of Council Inst. C.E., and seconded by Mr. R. P. Mea
Member of Council Inst. Struct. E., and was carried by acclamation.

Paper No. 5143.

"The Total-Heat–Entropy Diagram for Diphenyl."

By SIDNEY JOHN ELLIS, Assoc. M. Inst. C.E.

*(Ordered by the Council to be published with written discussion.)**

TABLE OF CONTENTS.

INTRODUCTION.

E binary-fluid engine proposed by Dr. Tremblay about 1850 has always d considerable theoretical interest for the student of thermodynamics. e outstanding practical application of the principle is to be found America, where a mercury-vapour turbine is expected to produce kilowatt-hour for a heat-expenditure of 9,500 B.Th.U. (corresponding a thermal efficiency of 35·9 per cent.). The disadvantages of mercury a working fluid are that it attacks metals other than steel, the poisonous ture of its vapour, and its cost. The first two appear to have been ercome in the American plant.

One working fluid which has been proposed instead of mercury is henyl, which is stated to be non-poisonous. It has been used as a at-transfer agent for many years in industrial chemistry. It has a high lting-point of 156° F., but a 3 : 1 mixture of diphenyl and diphenyl de melts approximately at 56° F†. The thermodynamic properties of dry saturated vapour of diphenyl have been published‡, and the thor has constructed the temperature–entropy diagram (*Fig. 1*, p. 228) m this data. Until quite recently no information has been available con- ning the specific heat of the superheated vapour, but the researches of W. S. Findlay have now provided this information for certain pressures

* Correspondence on this Paper can be accepted until the 15th March, and will ublished in the Institution Journal for October, 1939.—SEC. INST. C.E.

† W. S. Findlay, "Some Suggestions for Diphenyl Heat-Engine Cycles." *The er Engineer*, vol. xxix (1934), p. 89.

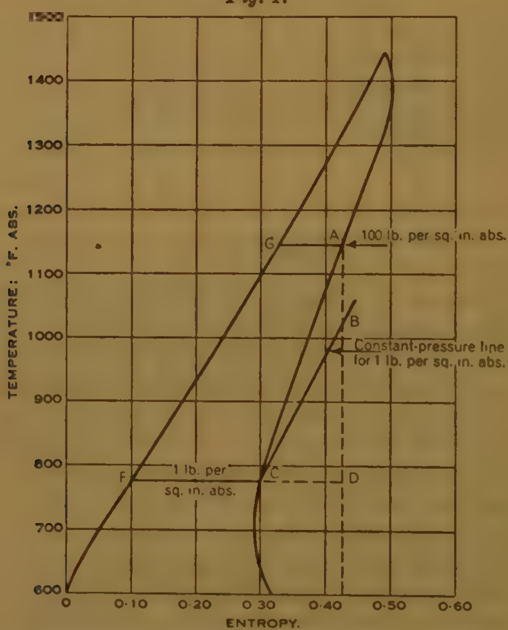
‡ H. M. Spiers, "Technical Data on Fuel," p. 144. London, 1935.

and superheats. Dr. Findlay has kindly placed his results at the dispo of the Author for the purpose of constructing a total-heat-entro diagram.

The Characteristics of Diphenyl.

One peculiarity of diphenyl may be mentioned. If the dry saturat vapour expands adiabatically it becomes superheated. This is in co trast to steam, which becomes wet if equilibrium is maintained. adiabatic-expansion line AD is shown in *Fig. 1* from a pressure of 100

Fig. 1.



Temperature–Entropy Diagram for Diphenyl

per square inch absolute to a pressure of 1 lb. per square inch absolu The line FCB is the constant-pressure line for the lower pressure.

The area CBD represents a thermodynamic loss entailed in any straig Rankine cycle (with an upper pressure corresponding to AG, and a lov pressure corresponding to FCB), compared with that for a fluid havi properties similar to steam.

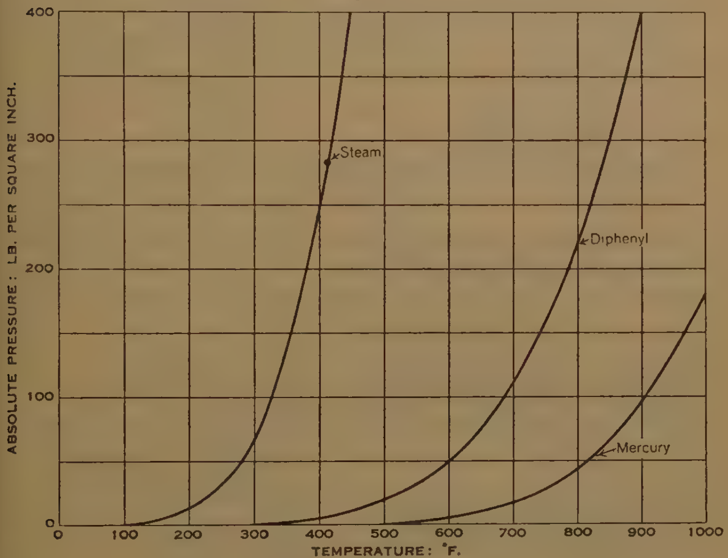
Mr. W. J. Kearton § pointed out that for an ideal fluid the ratio the latent heat of evaporation to the specific heat of the fluid should

§ "The Possibilities of Mercury as a Working Substance for Binary Fl Turbines." Proc. Inst.Mech.E., 1923 (Part II), p. 895.

great as possible. Mr. Kearton's figures for mercury and steam are own in Table I (p. 230), with the corresponding figures for diphenyl. rom these figures diphenyl appears to be at a disadvantage.

The vapour-pressure of the fluid is also a matter of interest, as although should exert a fairly high pressure at the lower temperature its vapour-

Fig. 2.



VAPOUR-PRESSURE CURVES.

ressure should not be too high at the higher temperature. The com-arison is made in Table II (p. 230) and in *Fig. 2.*

THE TOTAL-HEAT–ENTROPY DIAGRAM.

To obtain the necessary data to plot the total-heat–entropy diagram the superheat region, it was necessary to take the specific-heat data nd to make a step-by-step integration for the value of the entropy.

The values of the specific heat at intervals of 20° F. of superheat, ken from curves supplied by Dr. Findlay, are set out in Tables V to I, pp. 237 *et seq.*

To calculate the increments of total heat and entropy the arithmetic ean of the specific heat for each interval has been used.

Calling this mean value K_p, the increment of total heat $\Delta I = K_p \times 20$, nd the increment of entropy $\Delta\phi = K_p \log_e \frac{T_2}{T_1}$, where T_2 and T_1 differ by)° F.

TABLE I.

Substance:	Diphenyl.			Mercury.			Steam.		
Temperature: °F.	Latent heat of evaporation, L: B.Th.U. per lb.	Specific heat of liquid, C: B.Th.U. per lb.	Ratio, L/C.	Latent heat of evaporation, L: B.Th.U. per lb.	Specific heat of fluid, C: B.Th.U. per lb.	Ratio, L/C.	Latent heat of evaporation, L: B.Th.U. per lb.	Specific heat of liquid, C: B.Th.U. per lb.	Ratio, L/C.
200	175	0·41	426	131·34	0·0320	4090	977·8	1·0039	973
300	154	0·47	370	130·37	0·0328	3980	909·5	1·029	884
400	148	0·54	273	129·41	0·0329	3940	827·2	1·064	777
500	135	0·61	220	128·44	0·0332	3870	727·0	1·112	653
600	117·0	0·65	180	127·47	0·0335	3810	585·0	1·172	500
700	103·0	0·67	154	126·55	0·0335	3750	—	—	—
800	91·5	0·69	133	125·94	0·0341	3690	—	—	—

TABLE II.—VAPOUR-PRESSURES OF DIPHENYL, STEAM, AND MERCURY.

Temperature: °F.	Vapour-pressure of mercury: lb. per sq. in. abs.	Vapour-pressure of steam: lb. per sq. in. abs.	Ratio: $\frac{\text{Vapour-pressure of steam}}{\text{Vapour-pressure of mercury}}$.	Vapour-pressure of diphenyl: lb. per sq. in. abs.	Ratio: $\frac{\text{Vapour-pressure of steam}}{\text{Vapour-pressure of diphenyl}}$.
200	0·003729	11·52	3090	0·06	192
300	0·05351	67·0	1250	0·701	95·6
400	0·3984	247·1	620	4·20	58·9
500	1·863	679·0	364	16·30	41·6
600	6·625	1540·0	232	47·0	32·84
700	18·886	3099·0	164	110·1	28·10
800	45·343	—	—	221·8	—

Curvature and slope of the specific-heat curves between the intervals ll introduce an error in the increments. The Author feels, however, at the values so calculated are sufficiently accurate for a preliminary vestigation into the properties of this working fluid. The calculations e given in the Appendix (pp. 237 *et seq.*).

The superheat region of the total-heat–entropy diagram (Fig. 3, facing 232) has been plotted from the figures so calculated. Skew co-ordinates ere chosen to give the necessary displacement in the superheat-region, d some "fairing" of the curves in both the saturated and the superat-regions was found necessary.

Relative Efficiencies of Engines Working with Diphenyl and Steam.

Calculations have been made using the total-heat–entropy diagram to tain the efficiency of a diphenyl-engine and a steam-engine working tween the same temperature-limits (Tables III and IV (pp. 231–232)).

Table III.—Diphenyl-Engine.

ry saturated diphenyl. Conditions at condenser: pressure, 1 lb. per square inch absolute; temperature, 317·5 F.

ressure: lb. per sq. in. abs.	Temperature: °F.	Initial total energy, I_1: B.Th.U. per lb.	Total energy after adiabatic expansion, I_2: B.Th.U. per lb.	Superheat: °F.	Heat-drop, $I_1 - I_2$: B.Th.U. per lb.	Total energy of liquid at 1 lb. per sq. in. abs.: B.Th.U. per lb.	Heat expended: B.Th.U. per lb.	Efficiency: per cent.
00	784·3	453	370	306	83	70·2	382·8	21·68
00	687·5	397·69	336	240	61·69	70·2	327·49	18·8
50	606·5	354·24	308	188	46·94	70·2	284·04	16·20
30	554·7	330·7	290	150	40·70	70·2	260·5	15·50
14·73	491·5	301·8	270	107·5	31·80	70·2	231·6	13·80
5·0	411·45	266·4	247	58	19·4	70·2	196·2	9·9

Examples of the Use of Diphenyl.

The following three examples deal with diphenyl used in a binary-fluid tem, with steam as the second fluid.

ample 1.

Dry saturated diphenyl at a pressure of 200 lb. per square inch absolute pands adiabatically to a pressure of 1 lb. per square inch absolute, the perheat being taken from the exhaust vapour by heating the diphenyl its way to the boiler. The dry saturated diphenyl is then condensed, condenser acting as a steam generator without superheater. The am-turbine operates with a top pressure of 86 lb. per square inch abso-

TABLE IV.—STEAM-ENGINE.

Dry saturated steam working between the same temperature-limits as the diphenyl-engine referred to in Table III (p. 231). Final pressure, 86 lb. per square inch absolute; temperature, 317·5 F.

Pressure: lb. per sq. in. abs.	Temperature: °F.	Initial total energy, I_1: B.Th.U. per lb.	Entropy per lb. at initial condition, ϕ_1.	Entropy per lb. at final condition, ϕ_s.	Change in entropy per lb., $\phi_s - \phi_1$.	Temperature-change, T_5, corresponding to change $\phi_s - \phi_1$: °F.	Change in total heat, $I_1 - I_s$: B.Th.U. per lb.	Heat-drop: B.Th.U. per lb.	Heat expended: B.Th.U. per lb.	Efficiency: per cent.
2800	684·9	1058·9	1·1996	1·620	0·4204	327	−129·3	197·3	771·9	25·6
1600	604·74	1162·7	1·3265	1·620	0·2935	228	− 26·0	202·0	875·7	23·1
1100	556·28	1185·6	1·3765	1·620	0·2435	189	− 2·5	186·5	898·6	20·75
630	491·44	1201·5	1·4397	1·620	0·1803	140	+ 13·4	153·4	914·5	16·75
280	411·06	1201·8	1·5163	1·620	0·1037	80·5	+ 13·7	94·2	919·8	10·3

FIG: 3.

TOTAL-HEAT-ENTROPY DIAGRAM FOR

ıte and a condenser-pressure of 0·5 lb. per square inch absolute, and the ater is heated from 80° F. to 317·5° F. externally.

iphenyl.

At 200 lb. per square inch absolute, the temperature = 784·3° F., nd the total energy I_s = 452 B.Th.U. per lb.

At 1 lb. per square inch absolute, after adiabatic expansion, the emperature = 620° F., and the total energy I'_s is then 370 B.Th.U. er lb.

Hence the heat-drop = 452—370 = 82 B.Th.U. per lb.

The total energy after expansion = 370 B.Th.U. per lb., and the total nergy for dry saturated diphenyl at 1 lb. per square inch absolute = 222·45 .Th.U. per lb.

Hence the superheat = 147·55 B.Th.U. per lb.

Also, the latent heat at 1 lb. per square inch absolute = 152·25 B.Th.U. er lb. and the total energy of the liquid at that pressure = 70·20 B.Th.U. er lb., whilst that of the liquid at 200 lb. per square inch absolute = 359·3 B.Th.U. per lb.

Hence the heat which has to be supplied to the diphenyl is :

To the liquid, 359·3—70·2=	289·1	B.Th.U. per lb.
„ latent heat,	93·0	„
Total .	382·1	„

f this, regeneration gives 147·55 B.Th.U. per lb.

Hence the gross heat to be supplied = 382·1—147·55 = 234·50 .Th.U. per lb.

team.

Initial pressure = 86 lb. per square inch absolute dry and saturated.

Condenser-pressure = 0·5 lb. per square inch absolute.

The adiabatic heat-drop = 316·13 B.Th.U. per lb. and the latent eat = 896·7 B.Th.U. per lb.

$$\text{Hence, lb. of diphenyl per lb. of steam} = \frac{896{\cdot}7}{152{\cdot}25} = 5{\cdot}90.$$

The external heat which has to be supplied, heating the feed from 80° F. 317·5° F. = 239·53 B.Th.U. per lb.

eat Account.

Heat supplied externally :

5·9 × 234·5 = 1,380 B.Th.U. to the diphenyl, and 239·53 B.Th.U. to e steam. Total heat supplied externally = 1,619·53 B.Th.U.

Work done by 5·9 lb. of diphenyl = 5·9 × 82 = 488·52 B.Th.U.
" " 1 lb. of steam = 316·13 "
Total = 804·65 "

Hence the efficiency $= \frac{804 \cdot 65}{1{,}619 \cdot 53} = 49 \cdot 6$ per cent.

Also, lb. of diphenyl per total horsepower-hour $= \frac{2{,}545}{804 \cdot 65} \times 5 \cdot 9 = 18\cdot$

and lb. of steam per total horsepower-hour = 3·15.

Example 2.

Some of the superheat in the diphenyl after expansion is used to superheat the steam; the remainder is used for regenerative feed-heating. As before, the latent heat only of the diphenyl is used to evaporate water which has been heated to the boiling-point externally. The assumption is made that the steam is superheated to 620° F., the temperature of the diphenyl exhaust.

At a pressure of 86 lb. per square inch absolute and a temperature of 620° F., the total energy of the steam = 1,341·3 B.Th.U. per lb.; but the total energy for dry saturated steam at that pressure = 1,183·8 B.Th.U. per lb.

Hence the superheat represents 157·5 B.Th.U. per lb. Now, 5·9 lb. of diphenyl per lb. of steam are again required, and the diphenyl superheat available = 147·55 × 5·9 = 870 B.Th.U. per lb.; that required is 157·5 B.Th.U. per lb., and hence 712·5 B.Th.U. per lb. are available for regenerative feed-heating.

Heat Account.

Heat supplied externally:
(5·9 × 382·1) − 712·5 = 1,541·89 B.Th.U. to the diphenyl, and 239·5 B.Th.U. to the steam; total = 1,781·42 B.Th.U.

Work done by 5·9 lb. of diphenyl = 483 B.Th.U.

Work done by 1 lb. of steam at a pressure of 86 lb. per square inch absolute, superheated to 620° F., and expanded adiabatically to a pressure of 0·5 lb. per square inch absolute. = 383 B.Th.U.

Total work done = 483 + 383 = 866 B.Th.U.

Hence the efficiency $= \frac{866}{1{,}781 \cdot 42} = 48 \cdot 5$ per cent.

Also, lb. of diphenyl per total horsepower-hour $= \frac{2{,}545 \times 5 \cdot 9}{866}$
= 17·33,

and lb. of steam per total horsepower-hour = 2·93.

Example 3.

Operation is without regenerative feed-heating, the superheat and latent heat of the diphenyl being used to generate steam, and the water being heated externally to a temperature of 371·5° F.

Diphenyl.

Heat-drop = 82 B.Th.U. per lb.; total energy after expansion = 370 B.Th.U. per lb.; total energy of the liquid at a pressure of 1 lb. per square inch absolute = 70·2 B.Th.U. per lb.; hence the heat available = 299·8 B.Th.U. per lb.

The heat which has to be supplied externally to the diphenyl = 382·1 B.Th.U. per lb.

Also, lb. of diphenyl per lb. of steam $= \dfrac{896{\cdot}7}{299{\cdot}8} = 2{\cdot}99$.

Heat Account.

Heat supplied externally:

2·99 × 382·1 B.Th.U. to the diphenyl, and 239·53 B.Th.U., as before, to the steam, making a total of 1,381·9 B.Th.U.

Work done by 2·99 lb. of diphenyl = 2·99 × 82	=	245·18 B.Th.U.
,, ,, 1 lb. of steam	=	316·13 B.Th.U.
Total work done	=	561·31 B.Th.U.

Hence, the efficiency $= \dfrac{561{\cdot}31}{1{,}381{\cdot}9} = 40{\cdot}6$ per cent.

Also, lb. of diphenyl per total horsepower-hour $= \dfrac{2{,}545}{561{\cdot}31} \times 2{\cdot}99$

$= 13{\cdot}56$,

and lb. of steam per total horsepower-hour = 4·53.

Taking a boiler-efficiency of 85 per cent., a turbine thermodynamic efficiency of 83 per cent., and a generator-efficiency of 96 per cent., the thermal efficiency of the plant is 49·6 × 0·96 × 0·83 × 0·85 = 33·58 per cent.

The overall thermal efficiency of the mercury-vapour plant is

$$\frac{3{,}411 \times 100}{9{,}500} = 35{\cdot}9 \text{ per cent.,}$$ where 3,411 B.Th.U. is the heat equivalent of 1 kilowatt-hour, and 9,500 B.Th.U. is the estimated heat-expenditure. From this it appears that the possibilities of diphenyl are almost equal to those of mercury.

The Stability of Diphenyl.

With regard to the stability of diphenyl, the Author has been unabl to find any information concerning diphenyl itself, but in a Paper b Mr. R. L. Hemdal, Jnr.* some information on this point is given respectin mixtures of diphenyl oxide and diphenyl known as " Dowtherm," and th following is abstracted from that Paper :

The rate of thermal decomposition of " Dowtherm " in the presenc of air is :

Temperature : ° F.	Decomposition per month of 20 working days of 24 hours each : per cent.
650	0·25 – 0·35
700	0·4 – 0·6
725	1·1 – 1·6
750	approx. 3·3 – 5·0
775	7·0 – 10·0

The period of service to be expected from " Dowtherm " at variou temperatures before purification is desirable is :

Mean temperature : ° F.	Time : months.
650	45 –60
700	25 –37
725	10 –14
750	3 – 4
775	1½– 2

Purification is, however, desirable when the degradation-products reac 15 per cent. The purification may be carried out by distillation in th boiler itself.

Since in the cycles chosen the lower pressure is 1 lb. per square inc absolute, there is a possibility of air leaking into the low-pressure side c the diphenyl system, and the Table showing the decomposition c " Dowtherm " in the presence of air may be applicable.

With regard to the cost of the working fluid, the price of diphenyl i stated to be about 10*d*. per lb., whilst that of mercury is about 3*s*. 8*c* per lb. It must also be borne in mind that, whilst the supply of mercur

* " Industrial Developments in Heat Transfer with Organic Compounds." Tran Am. Inst. Chem. E., vol. xxx (1933–34), p. 378.

comparatively limited, there appears to be no practical limit to the pply of diphenyl.

The Paper is accompanied by three sheets of diagrams from which e Figures in the text have been prepared, and by a series of Tables inted in the following Appendix.

APPENDIX.

Table V.

Absolute pressure=1 lb. per square inch ; Saturation-temperature=317·5° F. Total heat (dry saturated), I_s=222·45 B.Th.U. per lb. ; entropy (dry saturated), ϕ=0·299.

Superheat: °F.	Absolute temperature: °F.	Specific heat.	Total heat (superheated) I_s: B.Th.U. per lb.	Entropy, ϕ.
0	777·5	0·439	222·45	0·299
20	797·5	0·440	231·24	0·3102
40	817·5	0·443	240·06	0·3211
60	837·5	0·445	249·04	0·3219
80	857·5	0·449	257·89	0·3436
100	877·5	0·455	266·93	0·3540
120	897·5	0·460	276·09	0·3644
140	917·5	0·467	285·36	0·3746
160	937·5	0·475	294·78	0·3847
180	957·5	0·484	304·37	0·3949
200	977·5	0·493	314·14	0·4050
220	997·5	0·505	324·12	0·4151
240	1017·5	0·517	334·34	0·4253
260	1037·5	0·528	344·79	0·4355
280	1057·5	0·542	355·49	0·4457
300	1077·5	0·557	366·48	0·4560

TABLE VI.

Absolute pressure=5 lb. per square inch; saturation-temperature=411·45° F. Total heat (dry saturated), I_s=266·41 B.Th.U. per lb.; entropy (dry saturated), ϕ=0·3290.

Superheat: °F.	Absolute temperature: °F.	Specific heat.	Total heat (superheated), I_s: B.Th.U. per lb.	Entropy, ϕ.
0	871·45	0·463	266·41	0·3290
20	891·45	0·466	275·70	0·3396
40	911·45	0·470	285·07	0·3490
60	931·45	0·476	294·53	0·3602
80	951·45	0·483	304·12	0·3704
100	971·45	0·490	313·86	0·3805
120	991·45	0·500	323·76	0·3907
140	1011·45	0·514	333·9	0·4008
160	1031·45	0·527	344·31	0·4110
180	1051·45	0·540	354·98	0·4213
200	1071·45	0·555	365·89	0·4316
220	1091·45	0·570	377·24	0·4420
240	1111·45	0·582	388·76	0·4525
260	1131·45	0·595	400·53	0·4630
280	1151·45	0·610	412·58	0·4736
300	1171·45	0·625	424·90	0·4842

TABLE VII.

Absolute pressure=14·73 lb. per square inch; Saturation-temperature=491·5° F. Total heat (dry saturated), I_s=301·8 B.Th.U. per lb.; entropy (dry saturated) ϕ=0·358.

Superheat: °F.	Absolute temperature: °F.	Specific heat.	Total heat (superheated), I_s: B.Th.U. per lb.	Entropy, ϕ.
0	951·5	0·48	301·8	0·358
20	971·5	0·485	311·45	0·3685
40	991·5	0·490	321·15	0·3779
60	1011·5	0·490	331·00	0·3878
80	1031·5	0·503	340·98	0·3975
100	1051·5	0·512	351·13	0·4073
120	1071·5	0·524	361·49	0·4171
140	1091·5	0·540	372·13	0·4269
160	1111·5	0·555	383·08	0·4369
180	1131·5	0·570	394·33	0·4469
200	1151·5	0·591	405·94	0·4571
220	1171·5	0·609	417·94	0·4674
240	1191·5	0·626	430·29	0·4778
260	1211·5	0·692	442·97	0·4886
280	1231·5	0·657	455·96	0·4992
300	1251·5	0·670	469·23	0·5098

TABLE VIII.

Absolute pressure=30 lb. per square inch; saturation-temperature=554·69° F. Total heat (dry saturated), I_s=330·7 B.Th.U. per lb.; entropy (dry saturated), ϕ=0·3805.

Superheat: ° F.	Absolute temperature: ° F.	Specific heat.	Total heat (superheated), I_s: B.Th.U. per lb.	Entropy, ϕ.
0	1014·69	0·495	330·7	0·3805
20	1034·69	0·500	340·6	0·3902
40	1054·69	0·505	350·65	0·3999
60	1074·69	0·510	360·80	0·4094
80	1094·69	0·520	371·1	0·4189
100	1114·69	0·533	381·62	0·4285
120	1134·69	0·545	392·4	0·4381
140	1154·69	0·565	403·5	0·4478
160	1174·69	0·580	414·9	0·4577
180	1194·69	0·602	426·7	0·4677
200	1214·69	0·623	438·9	0·4778
220	1234·69	0·640	451·56	0·4882
240	1254·69	0·658	464·59	0·4986
260	1274·69	0·675	477·82	0·5092
280	1294·69	0·690	491·46	0·5198
300	1314·69	0·700	505·36	0·5305

TABLE IX.

Absolute pressure=50 lb. per square inch; saturation-temperature=600·52° F. Total heat (dry saturated), I_s=354·94 B.Th.U. per lb.; entropy (dry saturated), ϕ=0·399.

Superheat: ° F.	Absolute temperature: ° F.	Specific heat.	Total heat (superheated), I_s: B.Th.U. per lb.	Entropy, ϕ.
0	1066·52	0·52	354·94	0·399
20	1086·52	0·527	365·41	0·4088
40	1106·52	0·534	376·02	0·4185
60	1126·52	0·546	386·82	0·4282
80	1146·52	0·562	397·90	0·4380
100	1166·52	0·585	409·37	0·4479
120	1186·52	0·612	421·34	0·4581
140	1206·52	0·638	433·84	0·4686
160	1226·52	0·662	446·84	0·4793
180	1246·52	0·687	460·33	0·4902
200	1266·52	0·706	474·26	0·5014
220	1286·52	0·725	488·57	0·5126
240	1306·52	0·740	503·22	0·5239

TABLE X.

Absolute pressure=100 lb. per square inch ; saturation-temperature=678·57° F. Total heat (dry saturated), I_s=397·69 B.Th.U. per lb. ; entropy (dry saturated), ϕ=0·426.

Superheat: ° F.	Absolute temperature: ° F.	Specific heat.	Total heat (superheated), I_s: B.Th.U. per lb.	Entropy, ϕ.
0	1147·57	0·54	397·69	0·426
20	1167·57	0·546	408·50	0·4354
40	1187·57	0·560	419·63	0·4446
60	1207·57	0·582	431·05	0·4542
80	1227·57	0·621	443·07	0·4641
100	1247·57	0·668	455·95	0·4745
120	1267·57	0·697	469·6	0·4854
140	1287·57	0·725	483·5	0·4965
160	1307·57	0·750	498·5	0·5079

TABLE XI.

Absolute pressure=200 lb. per square inch ; saturation-temperature=784·63° F. Total heat (dry saturated), I_s=452·84 B.Th.U. per lb. ; entropy (dry saturated), ϕ=0·4605.

Superheat: ° F.	Absolute temperature: ° F.	Specific heat.	Total heat (superheated), I_s: B.Th.U. per lb.	Entropy, ϕ.
0	1244·63	0·55	452·84	0·4605
20	1264·63	0·56	463·84	0·4694
40	1284·63	0·576	475·3	0·4783
60	1304·63	0·615	487·21	0·4875
80	1324·63	0·686	500·21	0·4975
100	1344·63	0·745	514·56	0·5082

Paper No. 5186.

"A Suggested Basis of Comparison for the Efficiency of Steam Turbo-Generators and of Steam–Electric Generating Stations."

By JAMES FREDERICK FIELD, B.Sc., Assoc. M. Inst. C.E.

(*Ordered by the Council to be published with written discussion.*)[1]

TABLE OF CONTENTS

INTRODUCTION.

In 1898 The Institution published the Report of its Steam-Engine rials Committee,[2] in which it was recommended that the performance of a ondensing steam engine could be most fairly compared, for the purpose f measuring its efficiency of operation, with that of an ideal engine working n the Rankine cycle within the same limits of steam-condition. The hermal efficiency of the actual engine was calculated from the total heat upplied at a given load, and the efficiency of the corresponding Rankine ycle was obtained by a simple calculation, preferably from steam-tables. he ratio of these efficiencies was the criterion of engine-efficiency, and this tandard of efficiency was adopted practically all over the world. A simple ondensing steam turbine is also most appropriately compared with the leal Rankine engine. The ideal Rankine cycle is completely reversible ithin itself.

In more recent years the possibility of increasing the thermal efficiency f the multi-stage turbine by feed-heating with bled steam has been ppreciated, and the principle of inter-stage steam-reheating has simul- aneously been introduced in many cases, chiefly to overcome the practical ifficulties of expanding excessively wet steam efficiently in the lower

[1] Correspondence on this Paper can be accepted until the 15th March, 1939, and ill be published in the Institution Journal for October 1939.—SEC. INST. C.E.

[2] "The Thermal Efficiency of Steam-Engines." Minutes of Proceedings Inst. .E., vol. cxxxiv (1897–98, Part IV), p. 278.

pressure-stages of the turbine. The simple Rankine cycle is not a appropriate criterion for this type of engine.

It was recognized by The Institution, however, that such plant shou be comparable with a modified Rankine cycle, in which the ideas of stea reheating and feed-heating are introduced to a corresponding degree : t Report of the Heat Engine Trials Committee of the Institution [1] in 19 described modified Rankine cycles, in which both ideas were incorporat in an ideal engine in which the whole of the steam-flow is reheated at chosen intermediate pressure, and in which the whole of the stea required for feed-heating is extracted at a pressure corresponding to t specified final feed-water temperature. The 1927 Report indicated t method of calculating the efficiency of this modified Rankine cycle a of using it as a criterion of performance for a feed-heating and/or reheati turbine.

The ideal Rankine cycle, modified by a single stage of feed-heatin has, however, the disadvantage of not being completely reversible with itself, as was the original Rankine cycle, and so the calculated improv ment in thermal efficiency over the simple Rankine cycle is less than obtained in practice. The discrepancy lies in the idea of bleeding t steam in a single stage rather than in a multitude of stages, so chos that there is an infinitely small difference in temperature between the bl steam and the feed water at any particular point ; that is to say, t process is reversible. In practice, efforts are made to approach the co dition of reversibility by having the turbine bled in several stages, as ma as five stages being fairly common practice.

It is now suggested that the Rankine ideal engine should be modifi to allow both for steam-reheating and for feed-heating by extraction steam in an infinite number of reversible stages, and that this should used as the modern ideal standard. Whereas the original Rankine engi consists of a boiler, superheater, cylinder and piston, condenser, a feed pump, the modification consists in adding a re-superheater throu which the drop in steam-pressure is infinitely small, and an infinite numb of reversible feed-heating stages.

Thermodynamic Considerations.

The efficiency of the original ideal Rankine engine is almost exact equal to

$$\frac{\text{adiabatic heat-drop to condenser}}{\text{total heat received per lb. of steam}}.$$

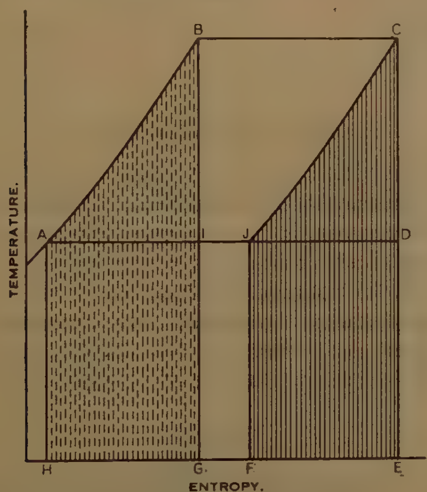
The error in this assumption, which neglects the work spent in t feed-pump, is very small, and can be ignored except for very high stea

[1] "Report of the Committee on Tabulating the Results of Heat Engine Trial The Inst. C.E., 1927.

essures. With dry saturated steam the cycle follows the first isothermal versible operation of the Carnot cycle, and likewise the second adiabatic versible operation, but in the third operation it proceeds to complete ndensation of the steam, and in the fourth the water is compressed in a ımp to boiler-pressure and external heat is added to bring it up to aporation-temperature, in place of the partial condensation and liabatic compression of the mixture of steam and water up to water at oiler-pressure and temperature, as required for the third and fourth erations in the Carnot cycle.

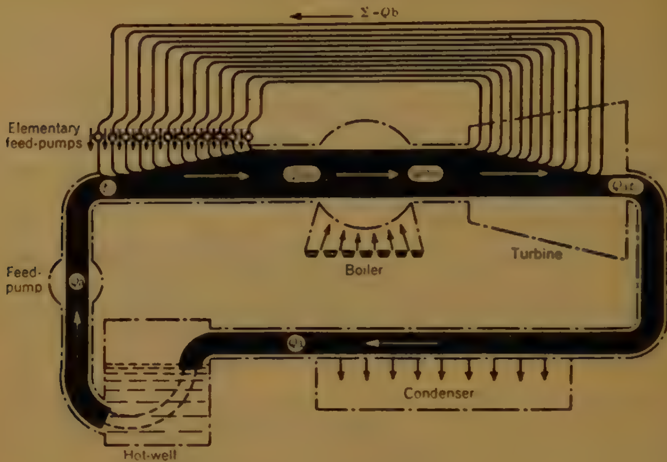
By feed-heating up to evaporation-temperature it is possible to avoid e taking-in of heat by the working fluid below this temperature. With turated steam the cycle is in effect a Carnot cycle, since the process of

Fig. 1.



ed-heating is assumed to be reversible, and the two engines work between e same limits of temperature. *Fig. 1* is a temperature-entropy diagram r this process. Assuming that each lb. of the working substance is heated om A to B by steam bled from the preceding lb., the work done per lb. ow through the engine will be area ABCDJIA — area JCDJ, or area BCJA, and the heat rejected externally will be area AJFHA. All the xternal heat is added at the upper temperature corresponding to B. he efficiency will then be $\frac{T_2 - T_1}{T_2}$, as in the Carnot cycle. *Figs. 2* and *3* . 244) show forms of such a reversible engine. In practice the greater art of the heat is added at the saturation-temperature corresponding to e boiler-pressure, but a limited amount of superheat can be added, and is heat, being taken in at a higher temperature, is more efficiently used.

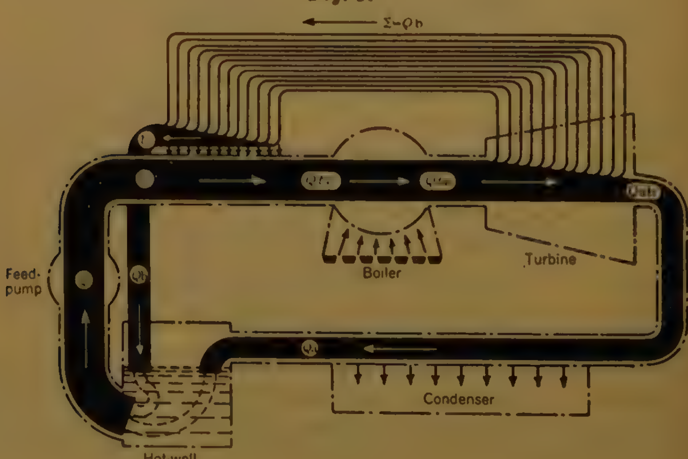
Fig. 2.



Practical Considerations.

In the application of feed-heating the boiler unit is required to furn a fixed quantity of heat per lb. flow of steam under certain conditions. F maximum efficiency the aim is the maximum conversion into mechani energy of the sum of the heat-drops available per lb. of the worki substance. There is, for instance, the expansion of the main body of ste to condenser-pressure, and of the several heater-quantities to intermedi pressures. Optimum use is made of the limiting steam-conditions arranging that heat is either rejected externally or transferred to the fe

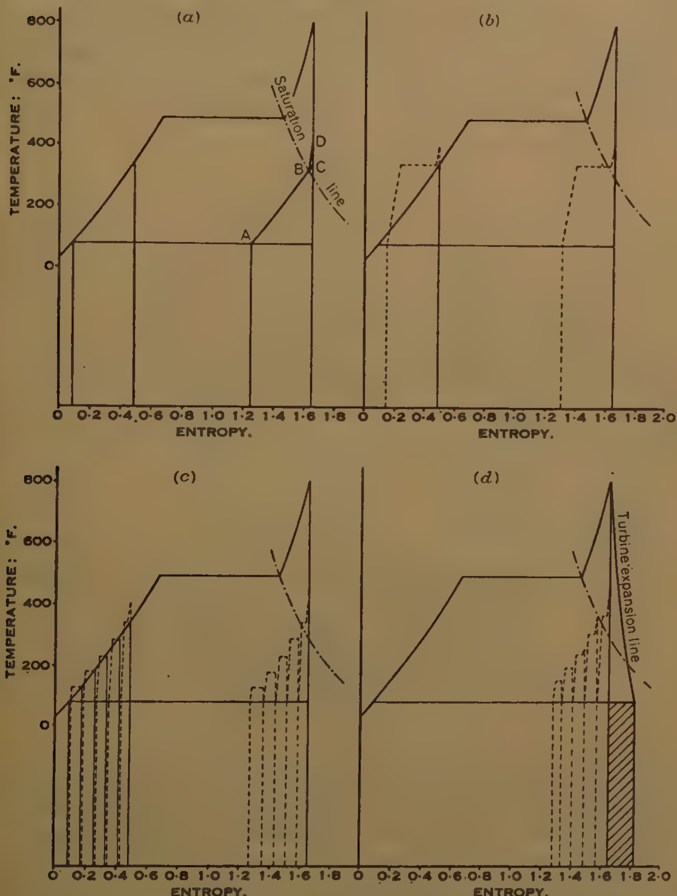
Fig. 3.



ater with the minimum difference in temperature ; that is, each part of he cycle approaches reversibility for maximum efficiency.

Steam is bled at a few intermediate points in the expansion, but at igher temperatures and pressures (and at a higher total heat) than it heoretically need have to transmit heat to the feed-water. The sur-

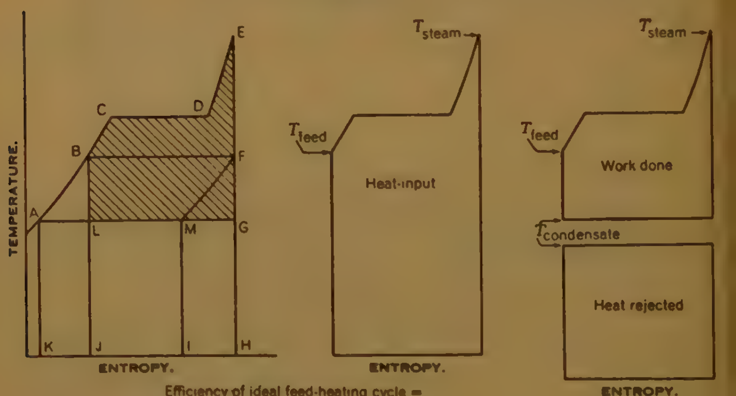
Figs. 4.



ce-type heater requires an economic minimum temperature-difference the order of 10° F. to transmit the heat through the resistant films at ch side of the heater-tube material. Expansion in the turbine is not rictly adiabatic, due to friction and shock, and reheat suffered by the eam due to this effect, being irreversible (*Figs. 4* (*d*)), reduces the efficiency the process. A lesser proportion of the stop-valve steam is bled off, and greater amount of heat is rejected to the condenser.

It is not convenient with steam bled in a superheated condition to he feed-water beyond the saturation-temperature corresponding to t pressure of the bleed-point. An examination of modern steam-conditio will show that the highest bleed-point theoretically takes place—and practice certainly would take place (*Figs. 4* (*b*), (*c*) and (*d*))—in a regi where the expanding steam is still in a superheated condition. The effe on the diagram-efficiency is shown by the line DB which bends to the li BA after passing the dry saturated region of expansion. Such an engi is not therefore reversible in the method of bleeding superheated stea Complete reversibility can only be achieved if the steam is bled at t temperature corresponding to the feed-temperature (line ABC, *Fi*

Figs. 5.



Efficiency of ideal feed-heating cycle =

$$\frac{\text{LBCDEG}}{\text{JBCDEH}} = \frac{\text{JBCDEH} - \text{JLGH}}{\text{JBCDEH}} = \frac{(\text{heat supplied under feed-heating conditions per lb. of water}) - (\text{absolute temperature of condensate}) \times (\text{difference in entropy between initial steam and feed-water})}{(\text{heat supplied under feed-heating conditions per lb. of water})}.$$

$$\text{or} = \frac{(i_1 - i_s) - T_s \times (s_1 - s_s)}{(i_1 - i_e)} \,*$$

4 (*a*)), and the ideal engine would therefore require additional eleme to compress this superheated bled steam isothermally to the correspondi saturation-temperature and pressure before actual mixing with the fe water. It is convenient to imagine the existence of these where necessa so that the formula for calculating the ideal efficiency is simplified. *F* 4 (*b*) show the effect on the diagram-efficiency of bleeding all the steam one stage, and *Figs. 4* (*c*) the improvement with five bleed-points.

In the application of reheating it is advisable to bleed steam at turbine-outlet to the reheater, rather than after reheating, since the for process is more conveniently reversible, and only a proportion of steam passing through the turbine-throttle also passes through

* I.E.C. symbols; see B.S.I. No. 752—1937.

heater. This does not affect the validity of the following formula which sumes the whole of the throttle steam to pass through the reheater ince the ideal engine is completely reversible within itself), and steam n be bled either before or after the reheat-point provided that it is bjected to reversible compression or expansion if necessary before its at is added to that of the feed-water. Alternatively, an ideal cycle can assumed in which no bleeding takes place, but is completed with an iabatic, an isothermal and an adiabatic, as in the Carnot cycle. This ill have the same efficiency as any other reversible cycle working within e same limits of temperature.

Standard of Efficiency.

The efficiency of the ideal engine =

$$\frac{(\text{Heat supplied to engine}) - (\text{heat rejected to condenser})}{(\text{Heat supplied to engine})}$$

For the modified Rankine cycle, using superheated steam and feed-ating, *Figs. 5* show the formula to be as follows :—

$$\begin{matrix}\text{e ideal feed-} \\ \text{heating} \\ \text{efficiency}\end{matrix} = \frac{\begin{matrix}\text{(Heat supplied under feed-heating conditions per lb.} \\ \text{of water)} - \text{(absolute temperature of condensate)} \\ \times \text{(difference in entropy between initial steam and} \\ \text{feed-water)}\end{matrix}}{\begin{matrix}\text{(Heat supplied under feed-heating conditions per lb.} \\ \text{of water)}\end{matrix}}$$

$$= \frac{(i_1 - i_6) - T_5 \times (s_1 - s_6)^*}{(i_1 - i_6)}.$$

It can be calculated easily from steam-tables.

Example.—Let the steam-conditions be from 600 lb. per square inch uge and 850° F., to 29 inches vacuum, with feed at 350° F. Then e total heat of the steam initially = 1437·24 B.Th.U. per lb., and e entropy = 1·6573; the total heat of the feed-water at 350° F. = 321·7 Th.U. per lb., and the entropy = 0·5039; also, the absolute temperature the condensate at 29 inches vacuum = 538·5° F. Hence the cycle-

$$\text{ficiency} = \frac{1115{\cdot}54 - 538{\cdot}5 \times 1{\cdot}1534}{1115{\cdot}54} \times 100 = 44{\cdot}3 \text{ per cent.}$$

It will be observed that when the feed-water temperature coincides

* I.E.C. symbols; see B.S.I. No. 752—1937.

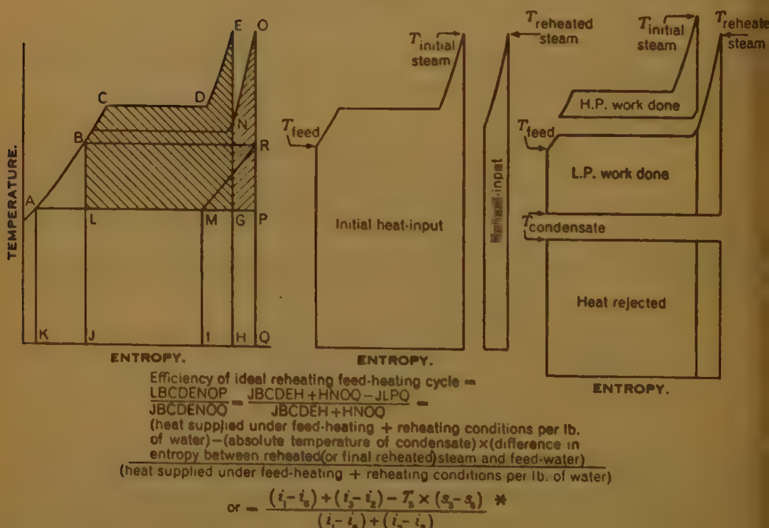
with the condensate-temperature the cycle-efficiency becomes that of t
Rankine cycle.

Figs. 6 illustrate the case of intermediate steam reheating, and fr
the diagram the formula is as follows :—

$$\begin{array}{c}\text{Ideal}\\ \text{reheating}\\ \text{feed-heating}\\ \text{efficiency}\end{array} = \frac{\text{(Heat supplied under feed heating + reheati conditions per lb. of water) — (absolute tempe ture of condensate) × (difference in entro between reheated (or final reheated †) steam a feed water)}}{\text{(Heat supplied under feed heating + reheating co ditions per lb. of water)}}$$

$$= \frac{(i_1 - i_6) + (i_3 - i_2) - T_5 \times (s_3 - s_6)^*}{(i_1 - i_6) + (i_3 - i_2)}.$$

Figs. 6.



Example.—Let the steam-conditions be from 600 lb. per square in
gauge and 800° F., reheating at a pressure of 115 lb. per squa
inch gauge to 800° F., to 29 inches vacuum, with feed at 350°
Then the total heat of the steam initially = 1410·15 B.Th.U. per lb., a
the entropy = 1·6362; the total heat after dropping adiabatically to
pressure of 115 lb. per square inch gauge = 1237·15 B.Th.U. per lb., a

† If more than one stage of reheating is adopted.
* I.E.C. symbols; see B.S.I. No. 752—1937.

ne entropy = 1·6362; adding 193·65 B.Th.U. per lb. as reheat to restore team to a temperature of 800° F., gives the total heat after reheating = 1430·8 B.Th.U. per lb., and the entropy = 1·8197; also, the absolute emperature of the condensate at 29 inches vacuum = 538·5° F., whilst the otal heat of the feed-water at 350° F. = 321·7 B.Th.U. per lb., and the ntropy = 0·5039. Hence the heat supplied = 1410·15 + 193·65 − 321·7 .Th.U. per lb. = 1282·1, and hence the cycle-efficiency

$$= \frac{1282{\cdot}1 - 538{\cdot}5 \times 1{\cdot}3158}{1282{\cdot}1} \times 100 = 44{\cdot}7 \text{ per cent.}$$

Application of the Formulas to Feed-heating Calculations.

Many engineers like to think of the adoption of steam-reheating or ed-reheating as giving a thermal efficiency gain of so much per cent. on ne equivalent Rankine cycle. This is probably due to familiarity with ne easily calculated Rankine cycle. There is also the practical considera-on that it becomes increasingly difficult and costly to maintain a specific andard of boiler-efficiency as the feed-temperature to the boiler rises eyond certain limits, and in high-steam-pressure developments a com-romise in feed-temperature has been generally necessary.

Below is given an efficiency-comparison for a feed-heating cycle and ne simple Rankine cycle, and for turbines of modern design working nder the corresponding steam-conditions. The steam-conditions assumed e :—

Initial pressure 570 lb. per square inch gauge and initial temperature 800° F., exhausting to 29 inches vacuum; feed-heating at maximum continuous rating to 340° F.

Using the formula the ideal-cycle efficiency = 43·62 per cent., and e corresponding simple Rankine-cycle efficiency = 38·76 per cent.; at is, an improvement is shown on the Rankine cycle of 12·54 per cent.

A steam turbo-generator set built for the above operating conditions, th five stages of feed-heating, has the following performance :—

Condensing heat-rate = 11041·15 B.Th.U. per kilowatt-hour generated.

Feed-heating heat-rate = 10013·91 B.Th.U. per kilowatt-hour generated.

Gain due to feed-heating = 10·26 per cent.

This compares with the ideal-cycle gain of 12·54 per cent.

APPLICATION OF THE FORMULAS TO TURBO-ALTERNATOR SETS AND POWER-STATIONS.

The complete heat-engine is a composite machine, consisting essential of a boiler, engine (turbine), condenser, feed-heaters, and a feed-pum In practice these are furnished by different contractors, and there has bee an inclination for the contractor to give a performance guarantee or a indication of the efficiency of his own portion of the plant, both to limit h responsibility and to enable him to gauge the excellence of his apparatu against that of competitors. Thus some turbine-makers, although givin heat-consumption figures for their turbo-generator sets with specifie condensing and feed-heating plant, mention the thermodynamic efficienc of the turbine only. Boiler-makers are concerned with the efficiency their boilers under specified steam-conditions.

The new formulas are convenient criteria either for the turbo-generat unit complete with feed-heaters, and extraction and boiler-feed pump bounded by the limits of the stop-valve, exhaust-flange, and high-pressu heater-outlet, or, alternatively, including the boiler and all the auxiliarie that is, for the station overall performance.

The efficiency-ratio so calculated for a station will serve as a criteri of the excellence of the detail-design and layout of the various items plant and the skill with which they are operated. Due allowance can made for operating conditions such as load-factor, circulating-water losse nature of fuel, etc.

The formulas have been applied to the performance of a steam turb generator of modern design : a 70,000-kilowatt machine works at stear conditions of from 570 lb. per square inch gauge and 800° F. to 29 inch vacuum, with feed at 340° F. The guaranteed heat-rate is 10,014 B.Th. per kilowatt-hour (excluding energy to the boiler feed-pump *). T ideal efficiency by the formula is 43·62 per cent., equivalent to appro mately 7,820 B.Th.U. per kilowatt-hour, so that the efficiency-rat for the steam turbo-generator only, including feed-heaters, is 78·1 p cent.

The formulas have also been applied to the performance of powe stations, and Mr. Johnstone Wright, M. Inst. C.E., Chief Engineer to t Central Electricity Board, at a recent meeting of The Institution†, co pared in diagrammatic form the performance of various power-stations recorded in the Electricity Commissioners' returns and in other publish

* The Author considers that since the boiler feed-pump is an essential part of t ideal engine of comparison, it should be included in the overall measurement o turbo-alternator.

† *Discussion on* "Constructional Work of the Fulham Power-Station," by J. Hay; and "Fulham Base-Load Power-Station: Mechanical and Electrical C siderations," by W. C. Parker and H. Clarke. Journal Inst. C.E., vol. 9 (1937–3 pp. 69 *et seq.* (June 1938.)

ata over the last 20 years. With his permission the Author is repeating
ıese figures in tabular form, with a note of the steam-conditions, etc.,
erived also from published data.

Station.	Date.	Steam-pressure: lb. per square inch gauge.	Total tempera-ture: °F.	Vacuum: inches mercury.	Feed-water tempera-ture: °F.	Ideal-cycle efficiency: per cent.	Actual overall efficiency: per cent.	Ratio of efficiency: per cent.
almarnock	1922	250	650	29	150	36·5	16·6	45·5
arton . .	1926	350	700	29	220	39·1	20·7	53·0
eptford West . .	1930	350	780	29	270	40·4	23·0	56·9
larence Dock . .	1933	400	700	29	270	40·5	24·8	61·2
unston "B" . .	1936	600	800*	29	350	44·7	26·87	60·1
attersea L.P. . .	1936	600	850	29	350	44·3	27·63	62·4
arking "B" . .	1936	600	800	29	350	44·0	26·54	60·3
ort Wash-ington, U.S.A. .	1937	1,230	825†	29	427	48·9	31·49	64·5
rimsdown "A" . .	1939	1,900	930**	28·75	375	49·0	?	?
attersea "B" . .	1940	1,350	950	29	400	49·3	?	?

* Reheating at 115 lb. per square inch gauge to 800° F.
† Reheating at 425 lb. per square inch gauge to 825° F.
** Reheating at 175 lb. per square inch absolute to 800° F.

Steam conditions are assumed to be those which obtain at the maximum continuous ıting of the turbine.

The Paper is accompanied by six sheets of drawings from which the 'igures in the text have been prepared, and by the following Appendix.

[APPENDIX.

APPENDIX.

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Paper No. 5138.

"Analysis of a Vierendeel Truss by Moment-Distribution and Deformeter Methods."

By EGBERT STEPHEN NEEDHAM, and
[L]EROY ARTHUR BEAUFOY, Ph.D. (Eng.), M.Sc. (Eng.), Assoc. MM. Inst. C.E.

(Ordered by the Council to be published with written discussion.)[1]

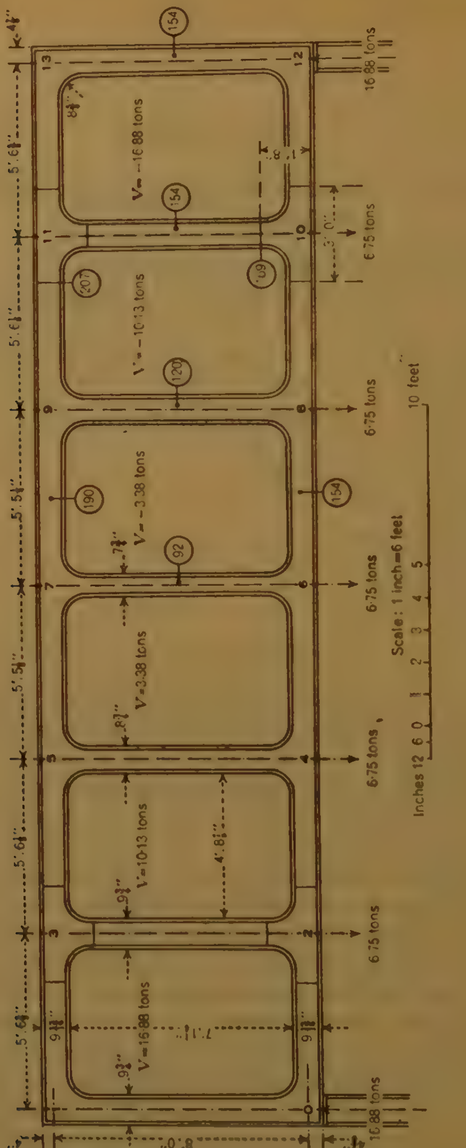
TABLE OF CONTENTS.

INTRODUCTION.

[Fo]r the design of the welded footbridge which was constructed by the [B]uilding Centre in New Bond street, London, to connect with the Grafton [G]alleries, for the purpose of extending their premises, the requirements [to] be met suggested the Vierendeel type of construction. The resulting [br]idge, which was erected in 1935, became the first welded Vierendeel [br]idge to be put up in Great Britain. Owing to a skew at one end of the [br]idge, the two trusses were not the same. The present Paper deals with [on]ly one of these trusses, and this had the form shown diagrammatically [in] *Fig. 1* (p. 254). It was symmetrical about mid-span and was built up from [pl]ates to give members of I section throughout. To increase the section-[m]odulus at the ends of the vertical members next to the end posts, ½-inch [sti]ffening plates were added as shown in *Fig. 1*. The moments of inertia [in] inch⁴ units of all sections are given as ringed figures. The loads assumed [fo]r design-purposes included allowance for concrete casing, slab-floor [an]d roof, brick walls with tile facing, and a live loading of 1·5 cwt. per [sq]uare foot. For the designed panel-length of 5 feet 8 inches the total load [pe]r panel amounted to 13·5 tons, or to 6·75 tons per panel per truss. [Fi]g. 1 shows the panel-lengths adopted for construction, which differ [sli]ghtly from the designed layout based on panels 5 feet 8 inches long.

[1] Correspondence on this Paper can be accepted until the 15th March, 1939, and [wil]l be published in the Institution Journal for October 1939.—SEC. INST. C.E.

Fig. 1.



ELEVATION OF TRUSS.

A complete analysis of this structure was made theoretically, using the Hardy Cross method of moment-distribution. An experimental check was obtained by applying the method of deformeter analysis to a small-scale xylonite model of the truss, and the present Paper shows the comparison between the results derived from the two methods.

Mechanics of the Moment-Distribution Method.

In his Paper * to the American Society of Civil Engineers, Professor [ardy Cross did not discuss the mechanics involved, preferring to treat 1e application of the method as if it were a physical occurrence rather 1an one of a series of simultaneous equations solved by successive approxi- 1ations. He makes use of three " beam-constants " defined as follows :—

Fixed-end moment.—By " fixed-end moment " in a member is meant the moment which would exist at the ends of a member if its ends were fixed against rotation.

Stiffness.—" Stiffness," as herein used, is the moment at one end of a member (which is on unyielding supports at both ends) necessary to produce unit rotation of that end when the other end is fixed.

Carry-over factor.—If one end of a member which is on unyielding supports at both ends is rotated while the other end is held fixed, the ratio of the moment at the fixed end to the moment producing rotation at the rotating end is herein called the " carry-over factor."

The values of these beam-constants may be derived by several methods, 1t as the method of slope–deflexion is perhaps the best known, use will : made of the fundamental equations of members in flexure subject to d restraint, as stated in Bulletin 108 † of the University of Illinois 1gineering Experiment Station, namely :—

$$M_{AB} = 2EK\,(2\theta_A + \theta_B - 3R) \quad . \quad . \quad . \quad . \quad . \quad . \quad (1),$$

d

$$M_{BA} = 2EK(2\theta_B + \theta_A - 3R) \quad . \quad . \quad . \quad . \quad . \quad . \quad (2),$$

e various symbols being explained on p. 256. The quantities E and I e here considered as constant throughout the length AB, which is the se assumed for the frame under consideration. The application of oment-distribution analysis for members of varying moment of inertia outside the scope of this Paper, but the derivation of the beam-constants : such members may be found in the discussion of Professor Hardy oss's Paper, notably in the contribution by Mr. A. W. Earl.‡ Valuable bles and graphs are also available § to shorten the work. The notation the above equations is as follows, referring to *Fig. 2* (p. 256).

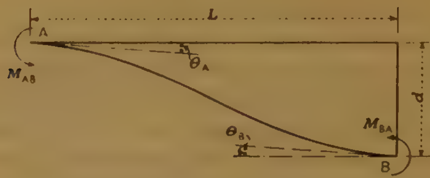
* " Analysis of Continuous Frames by Distributing Fixed-End Moments." Trans. ı. Soc. C.E., vol. 96 (1932), p. 1.

† W. M. Wilson, F. E. Richart, and C. Weiss, " Analysis of Statically Indeterminate uctures by the Slope Deflection Method." November, 1918.

‡ Trans. Am. Soc. C.E., vol. 96 (1932), p. 112.

§ *Discussion on* Messrs. L. H. Nishkian's and D. B. Steinman's Paper " Moments Restrained and Continuous Beams by the Method of Conjugate Points," by W. ppel. Trans. Am. Soc. C.E., vol. 90 (1927), p. 167.

Fig. 2.



Let M_{AB} denote the resisting moment at the end A of the memb AB.

„ M_{BA} „ resisting moment at the end B of the memb AB.

„ E „ modulus of elasticity.

„ K „ I/L where I denotes the moment of inertia the section.

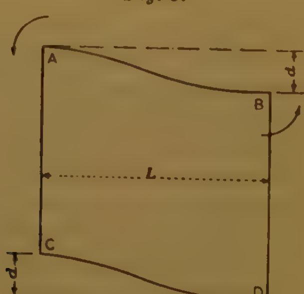
„ θ_A, θ_B „ the changes in the slope of the elastic curve at and B respectively.

„ R „ $\frac{d}{L}$, or the angular slope of the member in i deflected position.

Fixed-End Moment.

Let the end panel of the truss of *Fig. 1* be considered to be deflecte as in *Fig. 3*, with the joints fixed against rotation, as defined for fixed-en moment.

Fig. 3.



Then substituting zero for θ_A and θ_B in equations (1) and (2),

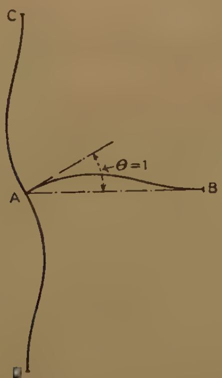
$$M_{AB} = M_{BA} = 2EK(-3R)$$

$$= -6EK\frac{d}{L} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

This is the familiar expression $\frac{6EId}{L^2}$ for end moment in the stanchions f a building frame subjected to lateral displacement. The negative sign for the resisting moment, the bending at the ends of members AB and D in *Fig. 3* being positive. As explained later, the moment at the end f a member is considered positive if it tends to cause clockwise rotation f the joint.

It will be noted that if d is the same for members AB and CD, the four orner moments in the panel are proportional to I/L^2. Assuming I to be he same for both members and L to be constant, then the four fixed end ıoments are equal. Their sum equals the shear in the panel multiplied y the panel-length, or, considering half the total panel shear to be carried

Fig. 4.



y each boom at its mid-point (point of contraflexure), the moments M_A, M_B, etc., equal

$$\tfrac{1}{2}V \times \tfrac{1}{2}L = \frac{VL}{4} \quad . \; . \; . \; . \; . \; . \; . \; . \quad (4)$$

When the shear V is known from the loads on the structure the fixed-nd moment can be written from equation (4) without reference to equaion (3), which involves the unknown deflexion d.

tiffness.

As defined above, stiffness of a member is the resistance to rotation or unit rotation at one end when the other end is fixed. Thus, in *Fig. 4*, et the three members meeting at A be subjected to a unit rotation of joint common to them all, the ends B, C, D remaining fixed. Equation (1) ives $M_{AB} = 2EK(2 + 0 - 0) = 4EK_{AB}$; $M_{AC} = 4EK_{AC}$; and $M_{AD} = 4EK_{AD}$. Thus any moment imposed on the joint A will be resisted y the several members in proportion to their K-values. For the ordinary

type of member with constant moment of inertia, the ratio I/L is all tha is implied in the term "stiffness," whilst the "stiffness-factor" of a membe for the end under consideration is the ratio which the stiffness of tha member bears to the sum of the stiffnesses of all the members at the join

Carry-Over Factor.

Due to the rotation at A in *Fig. 5*, equations (1) and (2) give for th resisting moments at the two ends, end B being fixed,

$$M_A = 2EK(2\theta_A + 0 - 0) = 4EK\theta_A$$

$$M_B = 2EK(0 + \theta_A - 0) = 2EK\theta_A$$

Thus for members of constant moment of inertia, the ratio of M_B t M_A is $\frac{1}{2}$, which is the carry-over factor as defined above.

MOMENT-DISTRIBUTION.

From a practical point of view the value of the moment-distributio method in solving "indeterminate" structures lies in not depending o the small rotations of joints and the angular slope ($R = d/L$) of member but in dealing directly with the moments themselves by simple arithmeti The procedure is unique in the initial fixing of all joints against rotatio Fixed-end moments are determined for all members and written down wit the proper sign. Any one joint is then released and rotation takes pla unless the algebraic sum of the moments at that joint equals zero, whic is not usually the case.

The algebraic sum of the moments around a joint is called the "u balanced moment," which, due to the rotation permitted, is distribute to the connecting members, so that the condition $\Sigma M = 0$ is establishe temporarily and the joint is again considered fixed against rotation.

The distribution of the unbalanced moment is accomplished by in posing on the joint an equal balancing moment of opposite sign allocate to the various connecting members in accordance with their respecti stiffness-factors. These moments having been written down, the su of all moments tabulated shows that the joint is now balanced.

The balancing moment thus imposed at one end of a member induc at the other end a moment of the same sign, as will be evident from study of *Fig. 5*. The carry-over factor being $\frac{1}{2}$, the moment written at t far end is one-half of the balancing moment just written at the near en and this is done for all the members of the joint just balanced.

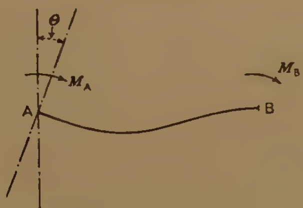
The moment carried over is added algebraically to the fixed-end other moments existing at the far end, and is thus included in determini the unbalanced moment of the next joint to be released. All the join are released in succession by the same procedure of distributing unbalanc moments and carrying over one-half to the adjacent (far end) join

ne process is then repeated once or twice until the unbalanced moments be distributed are negligible. Accurate balance of all joints may be cured, if desired, by sufficient repetition. The method is thus not an proximate one, but one of successive approximations. A summation all moments tabulated for each end of each member gives the final oments resulting at the ends of the members, and the assumed sections n be revised if necessary.

If the shears derived from the resulting moments are not equal to e known shears, a correction must be applied or the procedure must be tered to provide for maintaining the shears at their proper values.

This matter has been ably dealt with by Professor J. F. Baker, M.A., .Sc., Assoc. M. Inst. C.E., in his work for the Steel Structures Research

Fig. 5.



ommittee.[1] His method of tabulation for moment-distribution is com-endable. His sign convention has the same basis as that described erein, but unfortunately the positive and negative signs are used in the verse manner to those preferred by the Authors.

gn Convention.

In applying the moment-distribution method to the present analysis, e simple sign convention adopted was that positive moment at the end a member tends to rotate the adjacent joint clockwise.

By using this convention all members can be treated in exactly the me manner, and the necessity for employing a subsidiary convention, ch as that associated with turning the calculation-sheet through a right gle so that vertical members shall take up the position of horizontal embers, is eliminated. Carry-over moments have the same sign as the rresponding balancing moments, and the algebraic sum of all moments ound a joint in equilibrium is zero. For any given balance at a joint, e balancing moments are all of the same sign, their sum being equal d opposite to that of the unbalanced moment.

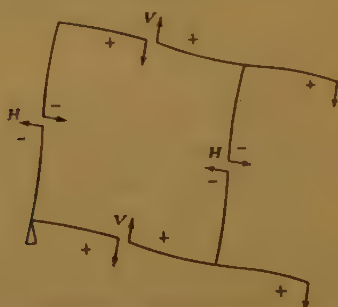
In *Fig. 6* (p. 260), representing the end panel of a truss, the shears V in e horizontal members tend to rotate the joints in a clockwise direction, the

[1] "The Stress Analysis of Steel Building Frames." Second Report of the Steel ructures Research Committee, pp. 214–220. H.M. Stationery Office, 1934.

moments at the ends of each member having the positive sign. T opposing shears H in the vertical members tend to rotate the joints in counter-clockwise direction, the resulting moments being negative an equal to the sum of the positive moments in the horizontal membe The signs give a mental picture of the type of curvature in the membe and once this convention is understood, the confusion about signs for rigi frame analysis disappears.

When the bending-moment diagram is drawn it is necessary to ado the usual convention, as shown in *Fig. 17* (p. 271), opposite signs no representing the tension or compression on one face of a member, whil the points of contraflexure also become evident.

Fig. 6.



The sign convention described is applicable to any problem in momen distribution, and it is hoped that those not familiar with it will try to u it in future, and that they will be rewarded for the slight effort involve

Calculations.

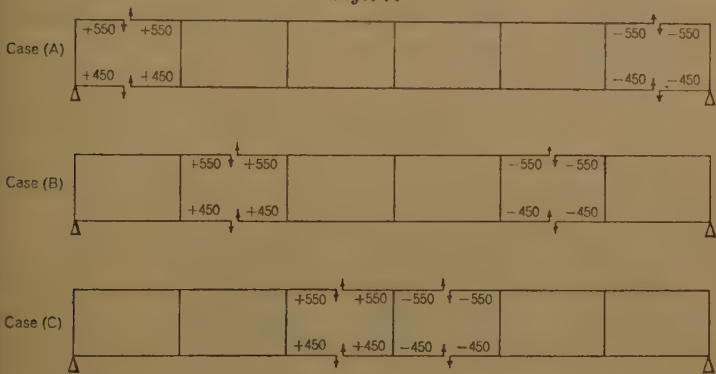
For the Vierendeel truss with parallel booms, the shears in the pan are resisted by the moments in the booms, the shear being equal to t sum of the boom-end moments divided by the panel-length, as noted abo (equation (4), p. 257). From the known shears the fixed-end moments f beginning the calculation can be written by this equation, assuming bo booms to have the same moment of inertia. Thus, in the end pan $\frac{VL}{4} = 16{\cdot}88 \times \frac{68}{4} = 287$ inch-tons is the fixed-end moment at each four boom-ends. However, as will be seen later in distributing the u balanced moments, a large proportion of the fixed-end moments is absorb by the verticals, due to the relatively large rotations of the joints, leavi to the booms only a portion of the moments which must be present equalize the known shears.

There is a method [1] of applying arbitrary joint-rotations which provi

[1] Hardy Cross and N. D. Morgan, "Continuous Frames of Reinforced Concret pp. 229–233 (*in particular*, footnote on p. 233).

itial moments of such magnitude that after distribution the correct ıears are realized. An advanced treatment [1] of the problem has been esented by Professor J. F. Baker, in which initial adjustments based ı deforming the structure for moment, shear, and thrust effects are eveloped and their relation to speeding up the convergence is discussed. ther methods of hastening the convergence and restoring the shear may esent themselves to the designer who has become familiar with moment-stribution, but in the analysis to be described a way of applying the

Figs. 7.



ardy Cross method was derived which avoids repeated corrections and iminates the uncertainties of the problem. It consists in noting the effect oduced on the truss by an arbitrary "unit shear" in each panel treated parately. By combining the resultant shears for each panel in three multaneous equations and solving, values were obtained by which the quired shears were satisfied and the final moments obtained.

The work was simplified in this case by the symmetry of the two halves the truss, which were treated simultaneously as shown in *Figs. 7*, and the assumption that the fixed-end moments for the upper and lower ooms might be taken in proportion to their relative stiffness, which rmitted treating each panel as a whole, and avoided the larger number equations necessitated by treating each boom individually.

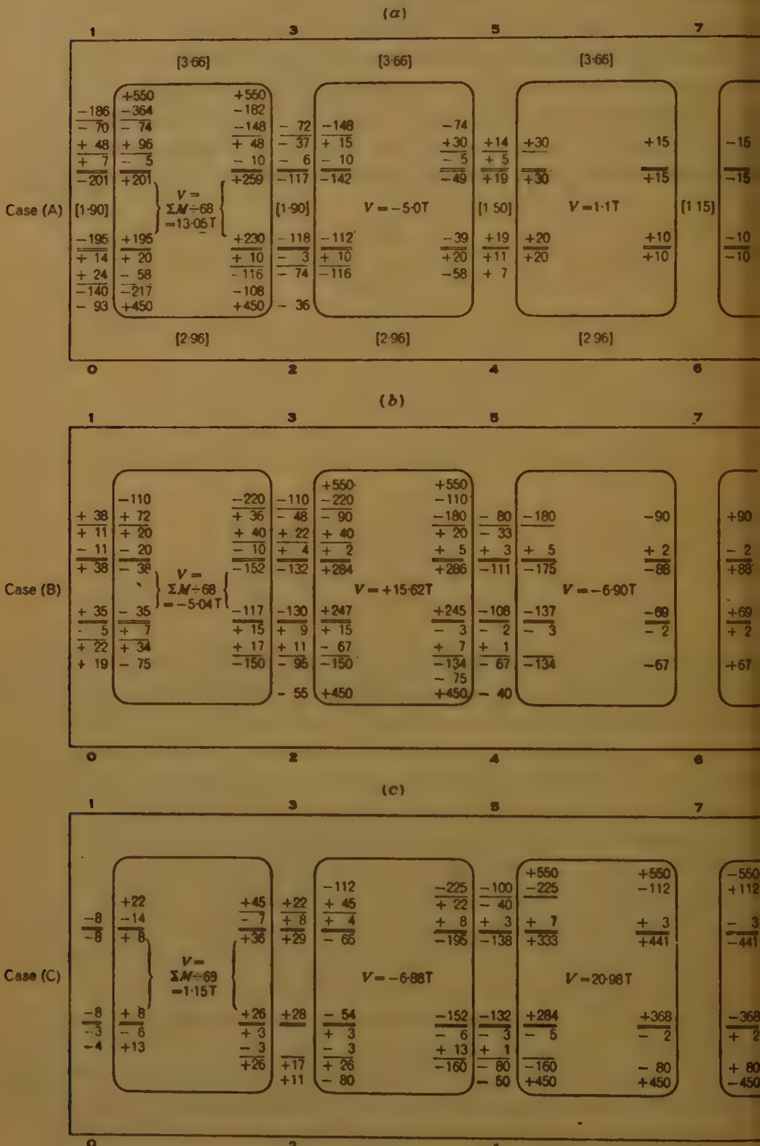
Taking 1,000 inch-tons end moment as a convenient measure of the unit shear" multiplied by one-half the panel-length, 550 inch-tons as proportioned to the upper boom and 450 inch-tons to the lower boom, r each of the three cases (A), (B), and (C) shown in *Figs. 7* and *8* (p. 262).

The complete distribution is shown in *Figs. 8*.

In *Figs. 8* (*a*) the stiffness of the members is given in brackets. In termining these values of I/L a shorter length than the distance between

[1] "A Distribution Method of Stress Analysis." Aeronautical Research Committee and M. No. 1667, 1935.

Figs. 8.



DISTRIBUTION OF 1,000 INCH-TONS UNIT SHEAR MOMENTS.

NOTE.—The totals (indicated by double lines) include the balancing operation, whi is omitted here for clarity. This accounts for the small differences in t additions and shows the rapidity of the convergence.

ntres of intersections was taken, to allow for local stiffness of the joint-
etail.

Demonstration of the Method.

In order to illustrate the application of the moment-distribution ethod, the calculation given in *Figs. 8 (a)* will be explained in detail.

The fixed-end moments for member 1–3 were assumed to be + 550 ch-tons at each end and + 450 inch-tons at each end of member 0–2. ll other members have zero moments.

Commencing at joint 1, the unbalanced moment is + 550. A balancing oment of — 550 is distributed to the two members 1–3 and 1–0 in pro- ortion to their stiffness- (I/L-) values 3·66 and 1·90, giving moments of - 364 and — 186 respectively. The algebraic sum of the moments now ritten at joint 1 equals zero, and a line is drawn below to indicate that the int is temporarily balanced. Care must be taken at this stage not to rget to carry over to the far ends of members 1–3 and 1–0 one-half of e distributed moments just written at joint 1. Thus — 182 is placed at int 3 and — 93 at joint 0. It should be noted that the sign of the oment carried over has not been changed. This is in conformity with e convention adopted, clockwise rotation of joint 1 tending to induce unter-clockwise rotation at joint 3, and at joint 0.

Passing to joint 3, the unbalanced moment is + 550 — 182 = + 368 ch-tons, which must be distributed to the three members 1–3, 3–2, and -5, in proportion to their stiffnesses 3·66, 1·90, and 3·66. This gives -148, —72, and —148, respectively, and the joint is balanced. Hence - 74 must be carried over to joint 1, — 36 to joint 2, and — 74 to joint 5.

Passing to joint 5, the unbalanced moment — 74 is distributed as - 30, + 14, and + 30, carrying over + 15 to joint 3, + 7 to joint 4, and - 15 to joint 7.

At joint 7 the reverse operation from the other half of the truss gives - 15 carried over from joint 9. The unbalanced moment therefore is ero, and there are no distributed moments to be dealt with or moments be carried over.

Passing to the bottom boom, the tabulation is written upwards for onvenience. At joint 0 the unbalanced moment is + 450 — 93 = + 357 d the distributed moments are — 140 and — 217, while — 70 is carried ver to joint 1, and — 108 to joint 2.

At joint 2, the unbalanced moment is + 450 — 108 — 36 = + 306, e distributed moments are — 116, — 74, and — 116, while the moments rried over are — 58, — 37, and — 58.

The distribution at joint 4 completes the first balance.

Reverting to joint 1, there now appear under the line unbalanced oments of — 70 and — 74 = — 144. The distribution is + 48 and + 96, d + 24 and + 48 are carried over.

The procedure is continued to complete the second balance, wit rapidly converging values. Finally, at joint 1, summing up the origin fixed-end moment, balancing moments, and carry-over moments, th algebraic totals indicated by the double lines are -201 and $+201$, an indeed for each joint it will be noted that $\Sigma M = 0$. It will also be note that in the panel where the loading was applied, the moments remainin in the top and bottom booms are less than half their original values, du to the elastic properties of the frame. The simplicity of the process will b at once realized if the above operations are followed through in detail.

The moments imposed on the first panel in *Figs. 8* (*a*) having bee balanced as demonstrated above, the resulting shears are obtained b dividing the sum of the four corner moments by the panel-length. I should be noted that the shear in the second panel is negative. The ca culation for cases (B) and (C) needs no comment, except perhaps to poi out in the case of (C) the effect of the simultaneous application of the fixe end moments at panel-points 7 and 6 derived from the adjacent panel As will be noted, the centre vertical (member 7–6) transmits no bendin stress, as there is no unbalanced moment to produce rotation.

The problem now is to determine what factor x to apply to the unit shea in each panel in turn in order to produce the required shears for th combination of (A), (B), and (C) for the panel.

Denoting the proportion contributed in each case by x_A, x_B, and x_C three equations are written as follows in which the constant terms are th required shears.

$$13{\cdot}05\ x_A - 5{\cdot}04\ x_B + 1{\cdot}15\ x_C - 16{\cdot}88 = 0 \quad . \quad . \quad . \qquad \text{(I)}$$
$$-\ 5{\cdot}0\ x_A + 15{\cdot}62\ x_B - 6{\cdot}88\ x_C - 10{\cdot}13 = 0 \quad . \quad . \quad . \qquad \text{(II)}$$
$$1{\cdot}10\ x_A - 6{\cdot}90\ x_B + 20{\cdot}98\ x_C - 3{\cdot}38 = 0 \quad . \quad . \quad . \qquad \text{(III)}$$

These equations yield the following values for the factors :—

$$x_A = 1{\cdot}81\ ;\ x_B = 1{\cdot}47\ ;\ x_C = 0{\cdot}55.$$

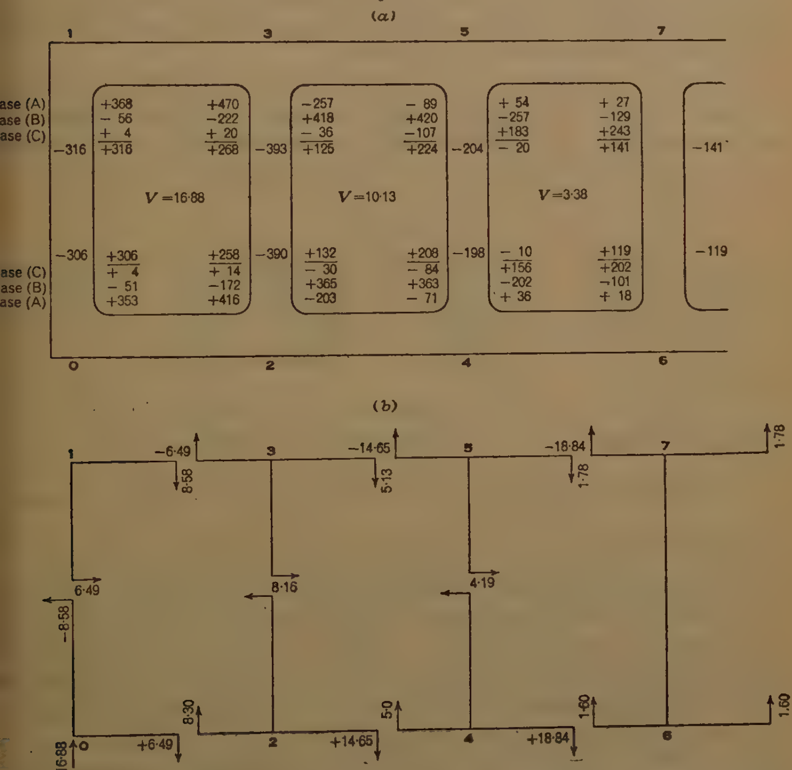
These factors are then applied to the boom terminal moments of (A (B), and (C), and a summation for final moments is made as in *Figs. 9* (*a* The moments for the verticals are obtained from these by inspectio The final shears in the members are determined from the sum (not th difference) of the end moments, divided by the panel-length in the cas of the booms and by the truss-depth for the verticals. The direct stress in the booms are the summation of the horizontal shears. The final diagra of forces is given in *Figs. 9* (*b*).

The end verticals transmit as a direct stress about one-half of the en reaction, depending on the relative stiffness of the two adjacent boom The intermediate verticals take direct stress only from the panel-poi load, which involves a small tension or compression depending on th load being applied at the bottom or at the top. If the panel-load is equall divided between top and bottom, the direct stress is negligible.

In the bending-moment diagram shown in *Fig. 17* (p. 271) the static gn convention used in the analysis is replaced by the diagrammatic onvention noted on p. 259. From this diagram the moment at the ritical section in the rounded corners is seen to be less than the calculated ioment at the intersection-points. For purposes of design 75 per cent. of he maximum moment was used, combined with any direct stress in the ember.

The unit stress was restricted by the specification to 8 tons per square

Figs. 9.



ach in tension. It is not proposed to enter into a discussion of permissible nit stresses for Vierendeel trusses, but it seems desirable to make this one omment: that this type of truss differs from that for which existing specifi-ations are applicable in that the deformation stresses are load-carrying tresses, and therefore the margin in present specifications to cover defor-nation or “ secondary ” stress should be allowed as an additional per-nissible working stress for the design of Vierendeel trusses.

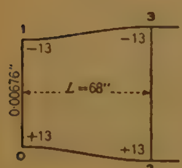
Effect of Axial Deformation.

The error due to the assumption that axial deformation is zero generally neglected in rigid-frame analysis. In Bulletin 108 of tl University of Illinois Engineering Experiment Station, to whic reference has been made[1], it is stated that "for steel frames of tl proportions common in engineering structures the error is well under 2 p cent."

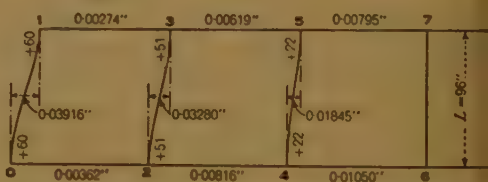
Although for the Vierendeel truss under consideration the deformatic due to axial loads in some cases increases the bending moments more tha 2 per cent., it will be shown that, for the purpose of design, the negle of axial-deformation stresses was justified.

The booms being designed primarily for bending, the direct unit stress are small, varying from ½ to 2 tons per square inch. The axial deformatio are given in *Figs. 10*. The relative displacement of the top and bottom vertical members is cumulative, the total for each panel being shown.

Figs. 10.



ADDITIONAL FIXED-END MOMENTS. END PANEL.



FIXED-END MOMENTS FOR HORIZONTAL DISPLACEMENTS.

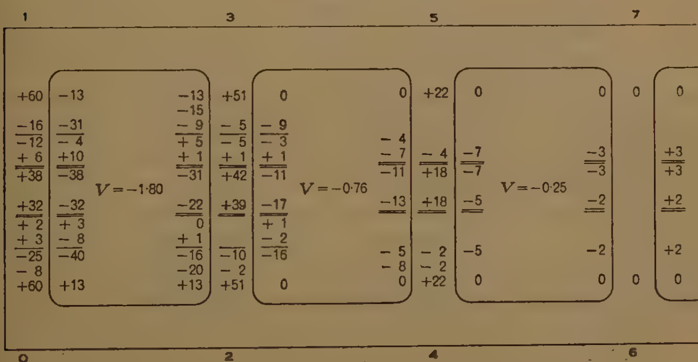
Fixed-end moments for the verticals derived from these displacemen by the formula $M = \dfrac{6EK\Delta}{L}$ are given in *Figs. 10* and *11*. As previous noted, the axial load in the intermediate verticals is negligible. The err in assuming for them zero vertical deformation is of the order of a fractic of 1 inch-ton bending moment. In the calculations all moments le than unity are neglected, and therefore in the fixed-end condition tl moments for the booms are given as zero, except for members 0–2 and 1– The load of 8·58 tons in the end vertical produces a shortening in it amoun ing to 0·00676 inch and this imposes on the adjacent boom membe fixed-end moments of 13 inch-tons, with negative sign for member 1- and positive sign for member 0–2 (*Figs. 10*).

Proceeding by the method of moment-distribution, the resultin shears shown in *Fig. 11* are obtained. It should be noted that the shea in the panels are negative or opposite to the shears resulting from tl external loading used in the analysis for the design (*Figs. 9* (*a*)). As n external loading is applied in considering axial deformation, the vertic

[1] Footnote (†), p. 255.

ears in the panels must be zero. Positive shears must therefore be posed on the negative shears of such value that a state of zero shear is stored. Having available the unit shear equations for positive shear, e values of x_A, x_B, and x_C given in *Figs. 12* (*a*) (p. 268) are determined substituting the shears of *Fig. 11* for the constant terms, and solving. pplying these x values to the moments of *Figs. 8* and summing up, the nding moments are derived for the restoring shears. *Figs. 12* (*a*) ve the moments corresponding to the two sets of shears, and their gebraic sum gives the moments corresponding to zero shear in the panels. hese are the moments attributable to axial deformation. They e of small proportions and the majority tend to decrease the total nding.

Fig. 11.

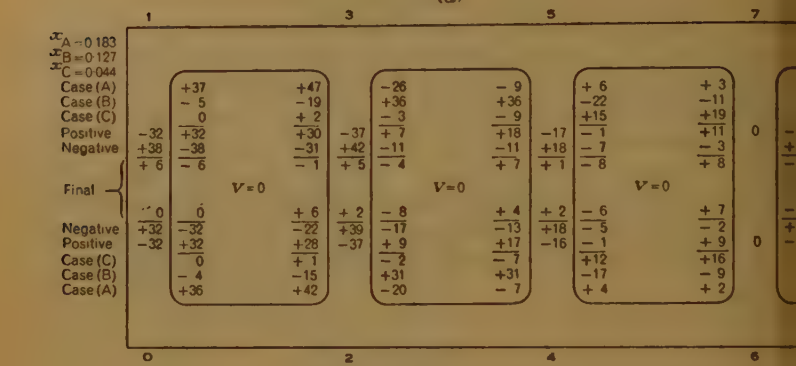


The distribution of the fixed-end moments of *Figs. 10* involves joint-tation only, as shown dotted in *Fig. 13* (p. 268), without deflexion of e truss as a whole. The restoration of zero shear is in effect the correc-on resulting from deflexion. *Fig. 13* is drawn to illustrate that in the eflected position the verticals take up a radial direction approximately right angles to the booms. It should be clear, therefore, that the dis-rtion of the truss due to axial deformation does not necessarily give rise large bending moments.

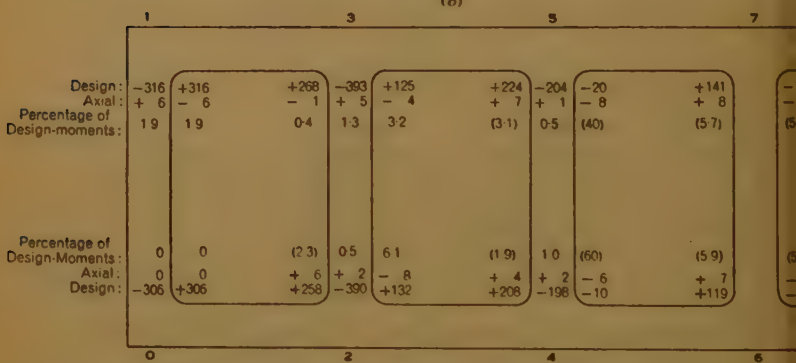
In *Figs. 12* (*b*) these moments are shown as a percentage of the moments sed for design. The boom sections having been selected for the bending panel-points 3 and 2, it will be noted that the maximum error calling r increased section amounts to only 2·3 per cent. At panel-points 5 and where the percentage of error is high, the moments involved are relatively nall. For this reason it may be expected that the members in the central anels will be sensitive to errors in calculation, and this is borne out in omparing with the results obtained in the model-analysis.

Figs. 12.

(*a*)



(*b*)

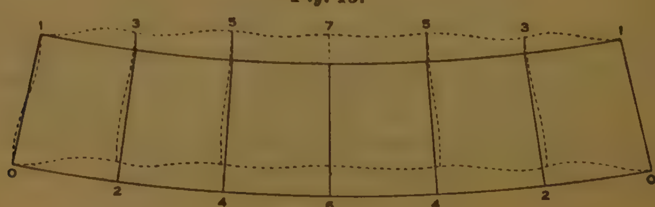


Note:—Percentages in brackets are increases in bending due to axial deformation.

The Deformeter Analysis.

In view of the assumptions necessarily made in order to obtain solution by moment-distribution, it was decided to apply the method deformeter analysis to a two-dimensional model of the truss, and thus obtain a mechanical solution which would be entirely independent of the assumptions.

Fig. 13.



It is not proposed in this Paper to enter into a discussion of the defor-
ıeter method as a method, as the principles on which it is based have been
ealt with elsewhere [1], whilst the particular technique adopted has been
lready described by one of the Authors [2]. Here it is intended to present
nd discuss the results of the model-test, with the addition only of brief
xplanatory notes.

The model used was cut to a scale of $\frac{1}{15}$ for the centre-line dimensions,
om a sheet of xylonite having a uniform thickness of 0·08 inch, whilst
ıe widths of the members were chosen so as to be proportional to the
iffnesses of the prototype members, the whole model being designed
as to satisfy as far as was possible the requirements of geometrical
milarity.

Small holes were drilled at the ends of the bottom chord, and in the
orking position the model, which was otherwise floating freely in a
orizontal plane, was secured about fixed pins passing through these
oles, thus reproducing the free end supports of the prototype.

The complete solution of the truss for bending-moment distribution
volved separate tests with the deformeter at 18 sections, chosen on the
rinciple of finding the bending moment at two points in each member
ith not more than a single cut per member, from which the values of the
ending moment at the centre-line intersections were deduced by assuming
linear variation of the bending moment along the whole length of the
ember from intersection-point to intersection-point. In practice these
aximum-values will not be attained, as it is evident that the moments
nnot be transferred from one section to another exactly at the points
intersection, but they do determine the variation of bending moment
ong the length of the member between the joints.

The terminal moments in the posts can be expressed in terms of the
rminal moments of the connecting chord members; thus, for example,
the upper end of vertical post No. 2 the terminal moment is $-(\beta_1 + \alpha_2)$,
d similarly at the lower end of vertical post No. 4 the terminal moment
$-(\delta_3 + \gamma_4)$, where α and β denote the terminal moments (left- and
ght-hand ends respectively) of the upper chord members, γ and δ having
similar significance in respect of the lower chord members; the suffixes
fer to the panel numbers, all numbering, both for panels and posts,
ing from left to right.

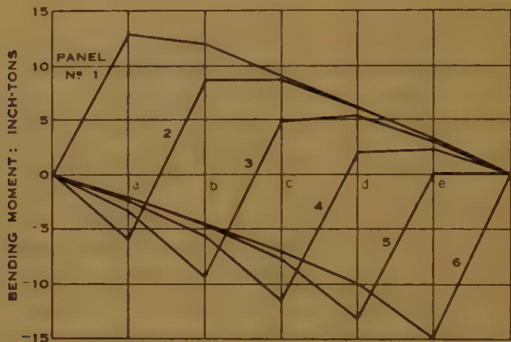
Examination of the experimental figures for terminal moments showed
ıall residual moments at the joints, but in all cases the discrepancies lay
thin the limits of accuracy of the method, the experimental figures being
markably consistent and quite suitable for use for all practical purposes
thout adjustment. From them were drawn *Figs. 14, 15,* and *16* (pp. 270-
1) showing the experimental influence lines for terminal bending moment

[1] "The Beggs Deformeter." *Engineering*, vol. cxxvi (1928), p. 31.

[2] L. A. Beaufoy, "The Stress Analysis of Rigid Frames by the Method of Elastic
odels." *The Welder*, vol. vii (1935), p. 498.

in the upper chord members and at the upper ends of the vertical post
The curves showing the variation in the maximum bending moment i
the lower chord members, and in the lower ends of the vertical post

Fig. 14.

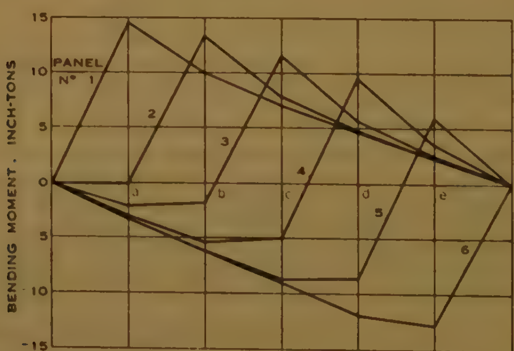


EXPERIMENTAL INFLUENCE LINES FOR TERMINAL BENDING MOMENTS α

as the unit load moves across the bridge span, are similar in form, an
are therefore not included here.

From these influence lines the bending-moment diagram for the enti
truss under the given conditions of loading was derived and is shown i

Fig. 15.

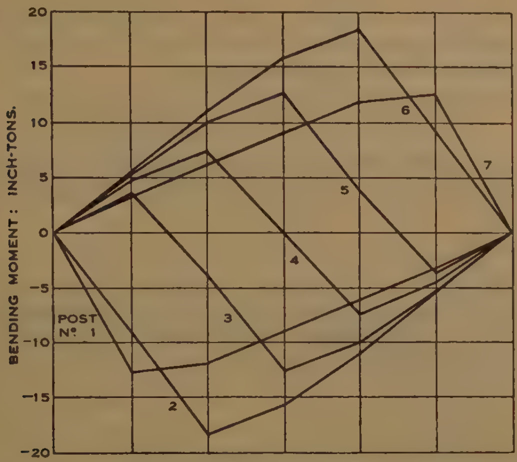


EXPERIMENTAL INFLUENCE LINES FOR TERMINAL BENDING MOMENTS β.

Fig. 17, where it is compared with the bending-moment diagram obtain
by the method of moment-distribution.

It will be seen that the agreement of the calculated results with tho
obtained by the experimental method is quite good. In making the co
parison it should be remembered that the calculated results were obtain

Fig. 16.



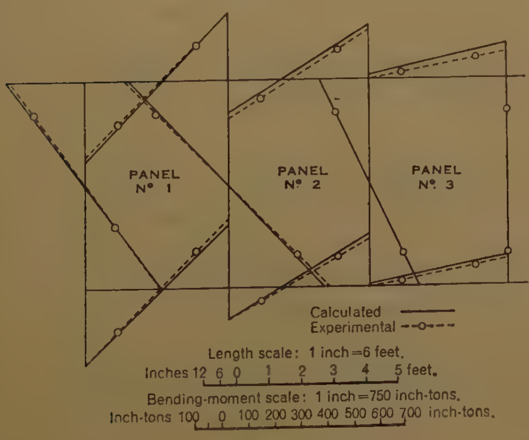
EXPERIMENTAL INFLUENCE LINES FOR BENDING MOMENTS AT UPPER ENDS OF VERTICAL POSTS.

ter making the following assumptions, none of which was required in e model-method :—

(1) that the imaginary unit shear applied to each panel in turn may be allocated to the top and bottom booms in proportion to their stiffnesses;

(2) that the effect of direct stresses on the distribution of moments may be neglected;

Fig. 17.



(3) that local rigidity at the joints due to gussets may be account for on an empirical basis by calculating the stiffness of t members, assuming a shorter length than the distance betwe centres of intersections; and

(4) that the depth of the members may be disregarded[1].

The Paper is accompanied by seventeen diagrams from which t Figures in the text have been prepared, and by one photograph.

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[1] For comparative figures illustrating the effect of this assumption alone in similar form of truss, *see Correspondence on* Mr. E. H. Bateman's Paper, "The Op Frame Girder." Journal Inst. C.E., vol. 1 (1935–36), p. 548*. (October 1936).

Paper No. 5136.

"Training of the Upper Nile."

By Frederic Newhouse, B.Sc. (Eng.), F.C.G.I., M. Inst. C.E.

Abstract.†

[Th]e Author refers to the original project of 1904 for securing complete [co]ntrol of the river Nile to provide for the needs of cultivation in Egypt. [Th]e works required were a dam at lake Albert, the training of the Bahr el [Ge]bel through or around the Upper Nile swamps, a dam on the White Nile, [a d]am on the Blue Nile near Sennar, and a dam at lake Tsana in Abyssinia.

The river loses nearly one-half its water in its passage through the [sw]amps of the Bahr el Gebel, and training works are projected to save as [lar]ge a proportion of this water as possible for use in Egypt. The water of [th]e White Nile is free from silt, making it possible to store water at all [sea]sons and stages of the river; this is not the case, however, on the Blue [Ni]le or the Main Nile, in which the early flood-waters are too muddy to be [sto]red without danger of silting up reservoirs.

The ultimate water-requirements of Egypt are put at 62,000,000,000 [cu]bic metres per annum at Aswan, distributed in definite monthly [qu]antities, plus the requirements of the Sudan at a concomitant stage of [de]velopment of about 6,000,000,000 cubic metres per annum, giving a [tot]al requirement of 68 *milliards* * at Aswan.

There is very little variation in the deficiencies of supply to requirements [in] the 6 summer months from February to July, which are always about [..] *milliards*, but the surplus in flood varies enormously, the flow of the silt-[fre]e late flood-waters ranging from about 7 *milliards* to about 18 *milliards*. [A] year such as 1925–26 with a total flow at Aswan of 70 *milliards* is con-[sid]ered as the "standard year"; such a year occurs about every 12 years, [an]d is defined as the worst year in which it is proposed to provide full [req]uirements of irrigation-water.

The problem of controlling the Nile is two-fold: firstly it is necessary [to] find storage for about 19 *milliards* compared with the present 8 [mil]liards, and secondly it is necessary to increase the flow in late flood, a [poi]nt that has been insufficiently stressed in the past.

Whilst it seems probable that water could be conserved by works on [lak]e Victoria and on the Victoria Nile, there are no data to enable engineers

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† The Paper has been published separately in the form of a book, copies of which [may] be obtained from Sir Isaac Pitman and Sons, Ltd., price 6*s*., postage 6d.—Sec. [Ins]t. C.E.

* 1 *milliard* = 1,000,000,000 cubic metres of water.

to estimate the amount or the cost. The Author shows from gener principles that the proposal to effect a permanent lowering of the level this lake is unlikely to lead to any useful result.

The Author then reviews the works proposed to be carried out, includi the turning of lake Albert into a reservoir 5,300 square kilometres in ar with a range of 10 metres, giving a total capacity of 53 *milliards.* T annual discharge is assumed to be 23 *milliards* of which 4·5 *milliards* will used during flood to maintain navigation, leaving 18·5 *milliards* to discharged during the summer, of which a part will reach Egypt.

There are three main projects for training the Upper Nile, two of whi have sub-variants. These are the Veveno–Pibor project for diverti water from the Bahr el Gebel above the swamps through the Vever Pibor, and Sobat rivers to the White Nile ; a similar diversion project fro Bor direct to the White Nile, either *via* Jonglei and following the Zera or in a straight line ; and the remodelling of the Bahr el Gebel itself, eith by enlarging the channel, or by confining it between banks that wou enclose its course without touching the channel.

The Author considers each of these projects in relation to their cost a the difficulty of carrying them out, and states that it is not improbab that the final choice may be a channel through the Sudd region wi embankments along the Bahr el Gebel.

Below these works are proposed a reservoir at Gebelein and a dam Gebel Aulia, with a cross cut connecting the Blue Nile above Sennar da with the reservoir at Gebel Aulia. On the Blue Nile, besides the Senn dam, the only conservation-work under consideration is a dam at la Tsana. The total storage capacity which would be available is shown be about 16 *milliards* compared with requirements of 19 *milliards ;* t possibility is therefore envisaged of scrapping the present Aswan da with its reservoir of 5 *milliards* capacity, and of building a new one to ho 8 *milliards*, thus giving the extra 3 *milliards* to bring the total storage up 19 *milliards*.

The Author sets out the costs of all the works proposed, which togeth amount to £E46,000,000. To utilize the water provided, about 3,000,0 *feddans*[1] in Egypt will have to be canalized, at a further cost £E45,000,000. The total cost of developing agriculture in Egypt to t final stage is therefore about £E90,000,000, whilst the return is put at t same value. The time required is controlled by the work of canalizi Egypt, and is estimated to take 75 years, the cost being spread over t period.

[1] 1 *feddan* = $1\frac{1}{25}$ acre.

Paper No. 5182.

"Indeterminate Structures: A New and Easy Mechanical Solution."

By Professor JEHANGIR ARDESHIR TARAPOREVALA, B.Sc.(Eng.), Assoc. M. Inst. C.E.

(*Ordered by the Council to be published in abstract form.*)[1]

r structures incapable of solution by the usual principles of statics, the ethod of using models for determining experimentally quantities such as nding moments possesses many disadvantages: the smallness of the odels; the effect of temperature-changes; the necessity for costly icroscopes; the fact that no visual aid is given to the designer; the very w readings that can be taken; the number of models required for the mplete solution of a complex frame; and the fact that all forces must in one plane, etc. In the Author's view his method obviates all these fects.

Let several members of a structure meet to form a rigid joint, and let be assumed that it is required to find the bending moment at the joint e to one of the members, under any condition of loading on the frame. ie Author's method is to remove the fixity of the member in question hinging it at the joint with the remaining members; the moment at e joint due to that member is therefore zero under all conditions of ading. He then fixes a pointer on the freed member and another on e of the rigidly-jointed members, as near as possible to the joint in each se; by hooking one of them, the ends of the two pointers are brought ose together, and made to register. On the assumed loading being plied to the frame the points will move out of registration on account of e relative movement of the freed member and the rigid group of members. ual and opposite bending moments are then applied externally on either le of the hinge, in order to bring the two pointers back into registration. ie applied moment is then the actual moment that would exist at the nt due to the member under consideration, if this member had been idly jointed instead of being hinged; that is, if the joint for all the mbers had been rigid.

The construction of the pointers is described, and the method of sighting d registering them is referred to.

The structures tested by the Author include continuous beams over o spans, portal-frames, a ring (with and without a central stiffening ce), arches, Vierendeel trusses, building frames, girders with forces in o planes, and a suspension-bridge. The results are set out in a series of bles and graphs, and are compared with theory.

[1] The MS. of the Paper may be seen in the Institution Library.—SEC. INST. C.E.

Paper No. 5181.

"Wave-Formation in Regulating Sluices."

By HASAN ZAKY, Ph.D., B.Sc., Assoc. M. Inst. C.E.

(Ordered by the Council to be published in abstract form.)[1]

THE Author points out that the flow through open-type river-regulati sluices is subject to periodic impulses. The occurrence of these impul is objectionable, as they may endanger the safety of the sluice-structure any of the following ways :—

(*a*) Whilst the water-level is at its crest in one sluice it is at its low in the next sluice, thus subjecting the dividing pier to alt nating side pressures which it was not designed to resist.

(*b*) In time of flood, with prevailing high water-levels, the crest the wave may attain such a height as to touch the bott of the gate, with the result that the groove may be brok or the structure may be damaged.

(*c*) The high velocity of the water may erode the cement pointi of the masonry.

(*d*) The waves cause a scouring effect on the floor of the structu just downstream of the piers.

The Author traces the cause of the periodic motion of the water to formation of vortexes downstream of the piers. He then outlines a ten tive theory to explain the formation and behaviour of the vortexes.

Some experiments on sluices of the Rosetta barrage are referred and are compared with a 1/25 scale model of three sluices of the Beh regulator; the comparison showed that the phenomena observed in full-size regulator are reproduced with accuracy in the model. The mod results also indicate that by streamlining the downstream ends of the pi the magnitude of the disturbances can be appreciably reduced.

The Paper is accompanied by five sheets of drawings and th photographs.

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[1] The MS. and illustrations may be seen in the Institution Library.—SEC. I C.E.

ENGINEERING RESEARCH.

THE INSTITUTION RESEARCH COMMITTEE.

Committee on Earth-Pressures.

E Railway Companies Association has informed The Institution that four main-line Companies and the London Passenger Transport Board ve decided collectively to contribute £1,000 per annum for a period of ears towards the cost of the research on soil-mechanics which is being ried out by the Research Committee.

The Committee is, of course, greatly encouraged, not only by this stantial contribution towards its funds, but also by the evidence thus wn that the railway companies appreciate the importance of the work. is felt that British engineers and the organizations that they advise are inning to realize the great importance of the study of soil-mechanics the solution of such problems as the power of various soils to support ds imposed upon them by heavy structures, and the estimation of the bility of slopes and retaining walls. It is confidently hoped that the mple set by the railway companies will be followed by other organiza-ns who have control of important earthworks, and that it shall no longer said that British engineers are less eager than those of other nations take advantage of the undoubted economies which a study of this portant subject is certain to bring about.

In order that this research may be more rapidly pushed forward, a cial Experiments Panel has been formed by the Research Committee, sisting of representatives of the railway companies and of the Building search and Road Research Stations, with one or two other members of e Institution interested in soil-mechanics, to supervise the experimental rk and report results to the Committee.

EXPERIMENTAL WORK ON HIGHWAYS.

The Report for 1937–38 of the Experimental Work on Highwa (Technical) Committee of the Ministry of Transport has now been pu lished[1]. Particulars are given therein of more than fifty sections of roa and footpaths on which a comprehensive scheme of practical testing being carried out. The investigations are grouped under six main headin —concrete, cement-bound macadam, tar and bituminous surfacing, th surfacing coats, surface dressings, and footpaths in rural areas—t objects of each test being discussed and full details given of the desig construction, and behaviour in service of the experimental sectio Costs of the sections are given where possible, though it is emphasiz that in many cases they would not be directly applicable to norm construction.

Special attention has been devoted to the durability of various typ of roads and surfacings. In order to obtain correlation between t behaviour of surfaces in service and in the road-testing machines at t Road Research Laboratory, similar sets of surfaces have been laid do on the Colnbrook by-pass and on the machines. Studies are being ma of the resistance to skidding of various surfaces, including wood block concrete, bituminous macadam, and surface dressings.

Cores cut from concrete roads continue to provide useful informatio it is considered that specifications for concrete roads should require co to be taken from the finished work, in order to check the thickness a quality of the concrete and the position of any reinforcement employed.

RESEARCH AT THE ROYAL SCHOOL OF MINES, IMPERIA COLLEGE, LONDON; NOVEMBER, 1938.

Three principal groups of investigations are being carried out at the Sch by the staff and post-graduate students, under the direction of Profes J. A. S. Ritson, D.S.O., O.B.E., M.C., B.Sc.; by Dr. M. A. Hogan and staff on behalf of the Safety in Mines Research Board; and by the Mini Research Laboratory of the British Colliery Owners' Research Associati (in collaboration with the Safety in Mines Research Board), under t direction of Mr. J. Ivon Graham, M.A., M.Sc.

In the first group, Assistant Professor B. W. Holman has now co

[1] H.M. Stationery Office, 2*s*. 6*d*. net.

ıded his experimental work on the design of magnetic separators with ıvelling magnetic fields, and all-iron and all-copper working models have en constructed. The effect of travelling electric fields on smokes and e particles has been studied by Mr. K. R. Gilbert. Mr. W. H. Wilson s developed a method of tri-dimensional mapping, which allows the urse and form of lodes, ore-bodies, etc., to be clearly visualized. The ccessive levels which are shown on ordinary mine-plans, employing ctangular co-ordinates, are replotted on a distorted graticule and sembled to form a block diagram. An ingenious and simple mechanical otter has been devised to effect the necessary transformation speedily, rking from a reduced copy of the ordinary plan. Mr. Wilson has also en engaged in tests of the accuracy of the precise tubular compass and shaft-plumbing instruments. The catalytic combustion of gases on tals, which is the basis of types of gas-detectors for use in mines, is ing studied by Dr. W. Davies.

Investigations for the Safety in Mines Research Board on supports and wire ropes are in progress, in charge of Dr. Hogan. Experiments have en made in methods of reinforcing pack-walls with light wire netting, d on the use of netting to enable walls to be built from very friable terial. In connexion with the underground pack-load measurements . W. H. Evans has studied the load-distribution on laboratory packs, d has made an investigation of the stability of flat arch-ribs. Mr. B. C. earn has carried out experiments on the strength of English-grown props m the new State Forests in Hampshire, and on filled and unfilled tubular el props. Studies of the causes of failure of winding and haulage ropes ve been carried on by Mr. A. E. McClelland and Dr. E. M. Trent. A w type of capel has been developed and has proved satisfactory in service. tigue and corrosion-fatigue tests on galvanized wires have shown wide ferences between the values of various types of galvanizing, and the ture of the steel surface before galvanizing and the microstructure of the ting have been found to be important; the work is being continued. ny tests and examinations of ropes, fittings, etc., that have failed in vice are made.

The third group of researches, in charge of Mr. J. Ivon Graham, is ncerned chiefly with problems of ventilation, health, and the prevention fires and explosions. A study of the production of coal dust in mining l of possible methods of minimizing it is in progress. The incidence and ıses of silicosis are being investigated in collaboration with the Pul-nary Diseases Committee of the Medical Research Council; it has been erved at the Banting Institute, Toronto, that nitrous fumes such as se produced by explosives accelerate the formation of silicotic nodules, l this may be related to the extra incidence of silicosis in anthracite ıes, where the coal-getting necessitates the use of more explosive than bituminous mines. The production of dust smaller than 5 microns by lling in various rocks is being studied; it has been found that a rock

containing 60 per cent. of free silica may give a fine dust containing or 33 per cent. of free silica, whereas fine dust from Rand "banket" m contain more than 70 per cent. The minimum fineness of stone-dust th is necessary to prevent coal-dust explosions is being studied. Atm spheric conditions in hot and deep mines and methods of cooling are bei investigated. Work on the breaking-out and detection of gob-fires being continued; sensitive methods of detecting an increase in the pr portion of carbon monoxide in the air have been found to give an ea indication of the danger of fire. The quantities and pressures of firedan in coals are being measured.

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